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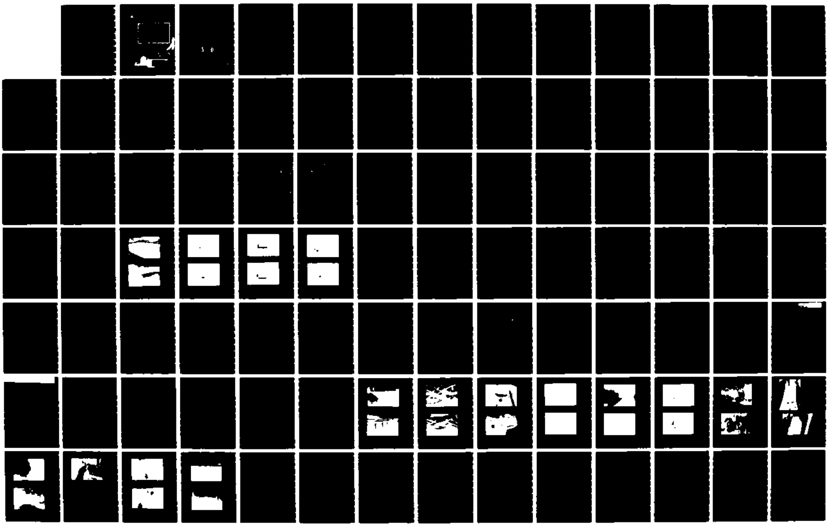

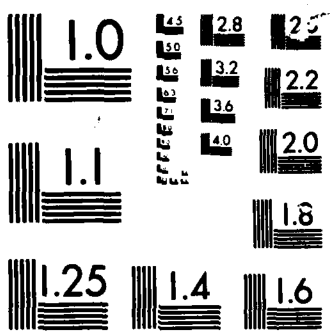

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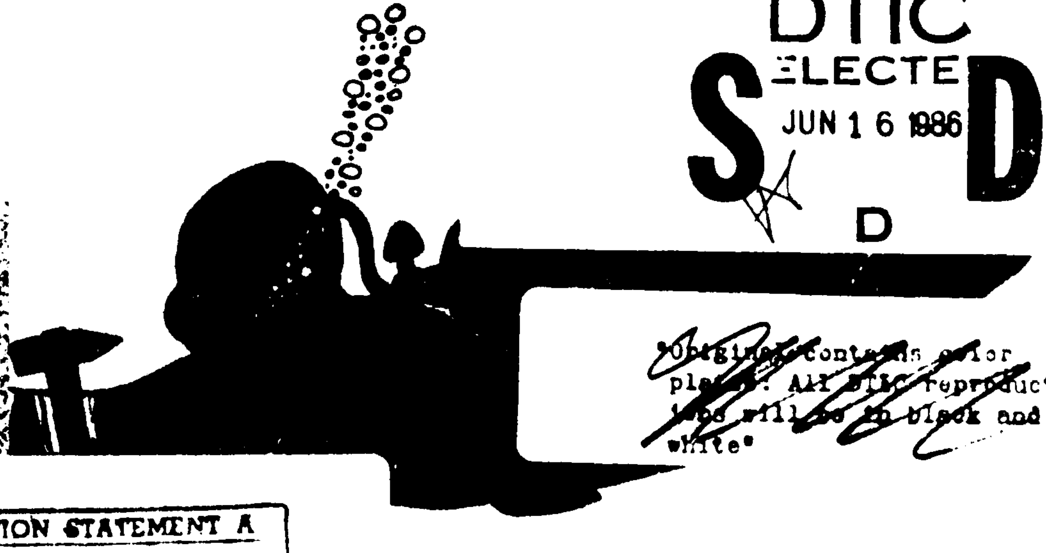
UNDERWATER FACILITIES INSPECTIONS & ASSESSMENTS

NAVAL AIR STATION,
NORTH ISLAND
SAN DIEGO, CALIFORNIA

FPO-1-84(19) OCTOBER 1984

OCEAN ENGINEERING AND CONSTRUCTION PROJECT OFFICE
CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON, D.C. 20374



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UNDERWATER FACILITIES
INSPECTIONS
AND
ASSESSMENTS
AT

NAVAL AIR STATION,
NORTH ISLAND
SAN DIEGO, CALIFORNIA

FPO-1-84(19) OCTOBER 1984

PERFORMED FOR:

OCEAN ENGINEERING AND CONSTRUCTION PROJECT OFFICE
CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON, D.C. 20374

CONTRACT: N62477-83-D-0190-0002
TASK 2

BY: BLAYLOCK-WILLIS AND ASSOCIATES
1909 MC KEE STREET, SAN DIEGO, CALIFORNIA 92110

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19. ABSTRACT (Continue on reverse if necessary & identify by block number)
An underwater facilities inspection was made of certain facilities at the North Island Naval Air Station, San Diego, California during the period July 20 to August 25, 1984. The assessed facilities were: (A) The Old Air Station Bulkhead (B) The Carrier Quay Wall or Bulkhead (C) Pier Bravo (Con't)

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BLOCK 19 (Con't)

(D) Pier J/K.

The Old Air Station Bulkhead is located at the northeast limit of the Naval Air Station. It was constructed in at least three separate increments. It appears that the first increment was constructed about the time that Pier J was built in 1921.

From the eastern limit of the wall adjacent to Building 316 to the vicinity of Building 29, the wall is composed of concrete T & G sheet piles with horizontal concrete wale and periodic tie-backs to dead men. To the West of Building 29, the wall appears to be a gravity retaining wall.

The tied-back wall has sustained sulphate damage which softened the surface concrete reducing the walls ability to sustain loads. It is recommended that the wall be strengthened if surcharges in excess of 200 pounds per square foot are planned close to the wall.

At various locations, the tied wall is leaking soil backfill creating surface cavities. These represent a hazard to traffic and should be filled.

The Carrier Quay Wall is located on the east side of the Naval Air Station adjacent to the City of Coronado. It was constructed in 1945 across the mouth of Spanish Bight, a shoal area which nearly cut off North Island from the peninsula.

Constructed of conventionally reinforced T & G sheet piling supported by a relieving platform, the wall has leaked backfill material into the bay from its inception. This leakage creates voids under the relieving platform thus admitting marine borers to untreated wood piles which support the platform. At various times during the life of the Quay Wall, attempts have been made to seal the wall. None have been completely successful. It is recommended that the wall be repaired using steel sheet piling installed a few feet outboard of the concrete sheet piles and the space between filled with lean concrete. The cost estimate for this method of repair is $5,408,000.

Pier Bravo is located at the west end of North Island adjacent to the entrance channel to San Diego Bay. It was constructed in two increments. The first increment consists of an access Pier 75 feet by 191 feet and a working Pier 75 feet by 625 feet arranged in a "T" shape. Its construction was finished in 1974. The second increment consists 327.5 foot both North and South of the working Pier and was built in 1979.

Pier Bravo is in excellent condition. It is recommended that it be inspected again in six years.

Pier J/K was the subject of an extensive underwater investigation conducted by Chesapeake Division in 1981. The present inspection is intended to reevaluate the assessments of damage and the recommendations of that report.

The Pier is located at the northeast corner of the Naval Air Station. It was constructed in three increments, in 1921, 1930 & 1958. The 1958 sections are in good condition but the piling (14, 16, and 18 inch square, conventionally reinforced) supporting the 1921 and 1930 sections are badly sulphate damaged.

EXECUTIVE SUMMARY

An underwater facilities inspection was made of certain facilities at the North Island Naval Air Station, San Diego, California during the period July 20 to August 25, 1984. The assessed facilities were:

A. The Old Air Station Bulkhead,

B. The Carrier Quay Wall or Bulkhead,

C. Pier Bravo, and

D. Pier J/K.

The Old Air Station Bulkhead is located at the northeast limit of the Naval Air Station. It was constructed in at least three separate increments. It appears that the first increment was constructed about the time that Pier J was built in 1921.

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1958. The 1958 sections are in good condition but the piling (14, 16, and 18 inch square, conventionally reinforced) supporting the 1921 and 1930 sections are badly sulphate damaged.

The Executive Summary of the 1981 report says in part:

"A condition of moderate to severe sulphate deterioration of the concrete was found in the piling to such an extent that it is recommended that the pier live load be restricted to 100 psf (pounds per square foot) and truck cranes in excess of 15 tons be prohibited.

The pier is adequately supported against earthquake forces (as defined in NavFac P-355) applied perpendicular to its prinicipal axis. However, the piles would not be expected to support the pier in the event of earthquake forces applied parallel to the principal axis.

It is recommended that useful life be considered no greater than five years."

This investigation reaffirms the recommendations of the 1981 report with the additional recommendation: that in the event that the pier is not demolished by the summer of 1986 the loads thereafter shall be limited to those imposed by small vehicles and small craft berthing with no crane loading.

NORTH ISLAND NAVAL AI

SAN DIEGO, CALIFO

EXECUTIVE SUMMARY

FACILITY	YEAR BUILT OR MODIFIED	NO. & TYPES OF PILE IN STRUCTURE	SIZE (AREA) FT²	SIZE (LENGTH) FT.
Old Air Station Bulkhead	1921 thru Unknown	1575 Piles in 1608 Ft.		4,580
Carrier Bulkhead	1945	2,200		3,525
Pier Bravo	1976 & 1979	300	73,410	
Pier J/K	1921 - 1930 - 1958	798	55,900	

ND NAVAL AIR STATION

EGO, CALIFORNIA

VE SUMMARY TABLE

SIZE (LENGTH) FT.	STRUCTURE	RECOMMENDATIONS	TOTAL REPAIR COST $
4,580	Reinforced Concrete Sheet Piles and Cast in Place Concrete	Limit Liveloads Adjacent to Sheet Piles to 200 lb. sq. ft. Fill Cavities in Soil Periodically as part of Normal Maintenance.	
3,525	Reinforced Concrete Sheet Piles	Repair by Sealing Face of Wall Against Soil Leakage.	$5,408,000
	24" Solid Octagonal, 20" Square and 18" Octagonal; All Prestressed Concrete Piles	Reinspect in 6 years.	
	14", 16" and 18" Square Conventionally Reinforced	Load After Mid 1986 be Restricted to Small Vehicles and Small Craft Berthing.	

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SECTION 1 - INTRODUCTION

1.1 CONTRACT DATA

Contract N62477-83-D-0190-0002 - Ocean Engineering Services in Support of Underwater Assessments at Various Locations.

This task required engineering services to document an underwater inspection and subsequently assess the integrity of the structural members supporting waterfront facilities at the North Island Naval Air Station, San Diego, California.

1.2 INTRODUCTION TO THE PROJECT

This inspection and assessment has been prepared under the Underwater Inspection Program conducted by the Ocean Engineering and Construction Project Office (FPO-1), Chesapeake Division, Naval Facilities Engineering Command, as part of NAVFAC's Specialized Inspection Program. It covers the inspection of Piers J/K and Bravo, the Carrier Quay Wall and the older Air Station Bulkhead at the northeast limit of the Station. The inspection of Pier J/K is considered to be a modified inspection, intended to update a complete inspection performed in 1981. The inspection was specifically oriented to the assessment of the physical condition of the concrete bearing piles of the Piers and the concrete sheet piling of the bulkheads. In addition, attention was directed to assessment of the magnitude of voids occurring under the relieving platform at the Carrier Quay Wall.

1.3 POST INSPECTION BRIEFING

Following standard practice in the Underwater Inspection Program, an exit briefing was given to Naval Air Station Staff Civil Engineer on 1 August 1984 by Mr. Wade Casey of Chesapeake Division, Naval

Facilities Engineering Command and Mr. A. J. Blaylock of Blaylock-Willis Associates. Attendees were Cdr. R. E. Herning, Staff Civil Engineer, NAVAIRSTA Code 18 and Mr. Scott Pogue, NAVAIRSTA Code 183. The observations of the inspection prior to structural analysis were provided as a "heads up" on the apparent overall condition of the facilities. Subsequent engineering analysis, as indicated in this report, have elaborated on these observations with no significant changes in the general conclusions.

SECTION 2 - ACTIVITY DESCRIPTION

2.1 LOCATION

The Naval Air Station, North Island, San Diego, California, is located on the east side of the entrance to San Diego Bay. It occupies the westernmost extension of the peninsula which includes the City of Coronado and the Silver Strand, and which provides the southern limitation of San Diego Bay. The Station comprises 2429.5 acres of land inboard of the mean high water line.

The dominant waterfront location of North Island with an unbroken shoreline more than eight miles in length, is particularly well suited to the complex mission of the Activity and its supported fleet and shore-based units. The coastal environment permits both direct air access to ships offshore without overflight of urban development, and convenient deep water access for berthing of aircraft carriers and other large ships.

2.2 HISTORY

The first government acquisition at North Island was made in 1893 when 18.05 acres at the extreme southwestern tip of the island were condemned for the construction of a jetty needed to protect the channel from siltation.

In 1901, an additional 38.56 acres adjacent to the jetty was condemned for the purpose of establishing a coast defense fort. Fort Pio Pico, a substation of Fort Rosecrans, remained until 1919. Its mission was the protection of the harbor entrance. This was the first military reservation on North Island.

In December 1910, Lt. Theodore G. Ellyson, the first Naval Aviator, was ordered to undergo flight training at the Curtiss Aviation Company on North Island. The Coronado Beach Company agreed to allow Curtiss to use North Island for a three-year period without cost. Curtiss, in 1911, constructed a landing field at the southeast side of North Island adjacent to the present location of Pier J/K.

The old Army/Curtiss dock, a wooden deck supported on wooden piles located at the northeast corner of the Station was torn down in 1921 to make room for Pier J, a reinforced concrete pier, supported on precast conventionally reinforced concrete piling. Most of this original construction remains today.

During the second World War and Korean conflict, North Island served the needs of the Naval Air Forces in the Pacific.

Since 1967, Naval Air Station North Island has continued as a very complex facility supporting fleet operations, training, repair and rework, supply and other activities of air and surface.

2.3 MISSION

The mission of the Naval Air Station, North Island is to maintain and operate facilities and provide services and materiel to support operations of activities and units of the Operating Forces of the Navy and other Activities, and Units as designated by the Chief of Naval Operations.

NAS North Island possesses a unique combination of physical features which would be impossible to duplicate elsewhere in California. A deep water port with carrier berthing adjacent to an air field in a climate where air operations are possible year long, are basic advantages and most useful for the performance of the Station's complex mission.

2.4 ENVIRONMENTAL DATA[1]

The climatic region of San Diego is classified as dry steppe (BSk) Kopen-Geiger classification system. The climate is characterized by ocean-influenced mild temperatures and light to moderate precipitation, primarily during the winter months.

The average annual rainfall recorded at Lindbergh Field one mile from the Naval Air Station is 10.4 inches. Heavy fogs occur in San Diego Bay approximately 24 days per year, most frequently in the Fall and Winter months.

Air temperature has an annual mean of approximately 63 degrees F. Coldest temperatures (45 degrees to 60 degrees) generally occur in January, and the warmest (68 degrees to 75 degrees) in August and September. Temperatures within the San Diego Bay immediate area are more moderate than in the surrounding upland areas.

Characteristic of the Bay area is the predominant sea-land breeze which persists as a westerly daytime wind, sometimes with a countering easterly land breeze at night. The average wind velocity at Lindbergh Field is 6.6 knots. Strong winds or gales are infrequent. The maximum wind recorded in San Diego occurred in November of 1944. It was from the southwest and 51 mph.

The larger San Diego area is subject to adverse meteorological conditions that are conducive to the concentration of air pollutants (smog). However, the Bay area experiences fewer air quality impacts due to the prevailing westerly winds and the absence of significant pollutant sources to the west.

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(1) Bibliography

San Diego Bay is crescent-shaped, about 22 miles long, and from 1/4 to 2-3/4 miles wide. It covers 18 square miles and contains 300,000,000 cubic yards of water at mean tide. The Bay tidal prism (the volume of water contained between high and low tide horizontal planes) is about 1/3 of its total volume.

Water depths in the northern section of the Bay generally exceed 30 feet, with about 70 feet maximum.

Average tidal range is 5.6 feet and extreme range is 10.0 feet. The maximum tidal currents at the facilities addressed in this report are less than 2 feet per second.

Prior to 1944 a shallow bay or gulf existed south of the present position of the Carrier Quay Wall. It was called Spanish Bight and was separated from the ocean on its southern extremity by a few hundred feet of sand deposit. This deposit was the only land connection of North Island to the Coronado peninsula. Principal traffic access to North Island was over a wooden pile supported causeway bridge in line with McCain Blvd. (4th Street).

In 1944 the construction of the Carrier Quay Wall was started along with the commencement of dredging operations which filled the Bight.

Historically, the Bay floor and margins are characterized by formational materials, sand, silt, clay and mud deposits. Mud deposits characterize eastern and southern margins of the Bay.

Past dredging activities have removed most of the mud deposits in the Naval Air Station area so that medium dense, silty sands are encountered a few feet below the existing bottom. The deeper deposits are quite dense and exhibit considerable structural competence.

The State of California is within an active seismic region. San Diego has experienced mild earthquakes in recorded history, but none have been catastrophic.

There are several fault systems in Southern California which must be considered in making a seismic assessment of the Naval Air Station for potential earthquake damage. These include the Rose Canyon and La Nacion Faults which are in the vicinity (three miles and two miles respectively), the Elsinore Fault located 50 miles to the east, the San Jacinto Fault 75 miles distant to the east, and the San Andreas Fault 85 miles to the east. It is understood that the largest probable magnitude earthquake would be generated by the San Andreas Fault (8.3 Richter scale). However, the San Jacinto Fault with a maximum probable magnitude of 7.8 could produce the largest ground acceleration in San Diego due to its closer proximity. That acceleration is estimated to be 20 percent g (gravity).

As described above, some of the Naval Air Station is reclaimed tidelands produced by dredged fill. These soils are susceptible to liquifaction in the presence of strong seismic energy waves, with resulting threat to existing structures.

Water quality in San Diego Bay is presently acceptable for most human activities, including water recreational purposes. In recent history, it has not always been this good. The first collection plant for area sewage was constructed by the City in 1887 to collect the random discharges that were polluting the Bay. The pollution had been so concentrated that the Navy had expressed concern that the Bay waters were affecting the paint on naval vessels. However, untreated and partially treated sewage continued to be discharged into the Bay by the surrounding communities until 1963.(2)

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(2) Bibliography

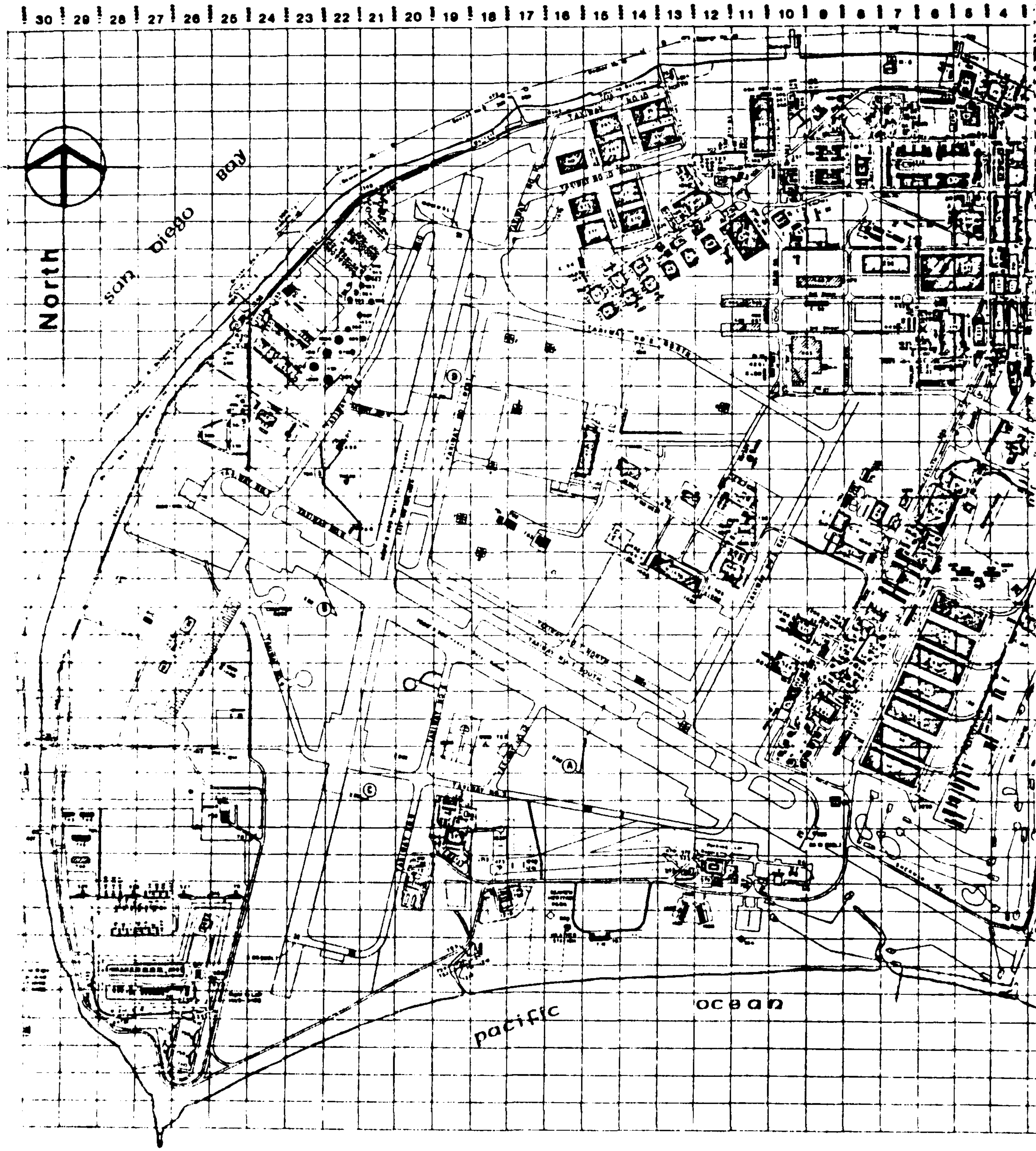
At that time, industrial and municipal sewage discharges were required to flow into the San Diego Metropolitan Sewage System. This system discharges its effluent into the ocean west of Point Loma.

The concentration of sulphate ion in open ocean water is high enough to create an environment hostile to Portland cement concrete. (See Section 5.2).

Pier J/K and the adjacent Air Station Bulkhead were exposed to the higher concentrations of sulphate ion for over thirty years due to the bay sewage problems. As a result they exhibit considerable sulphate damage. The Carrier Quay Wall exposed to only twenty years of polluted sea water and probably constructed with Type II cement shows small evidence of sulphate attack. But it is not considered significant. Pier Bravo shows no evidence of sulphate exposure.

Marine vegetation exists *within* San Diego Bay in the forms of various species of algae and one species of sea grass. The sea grass grows in the calm water near shore areas adjacent to the Naval Station. Marine algae are represented by large filamentous forms of red and green algae such as witche's hair or mermaid's hair. In addition, forms of green algae such as sea lettuce are found attached to rocks and marine structures. Over 200 species of marine invertebrates have been found. Sediment samples reveal infaunal organisms, including many species of polychaetes, small crustaceans and various bivalves.

Marine invertebrates found on pier piling, rocks, and marine floats include lobsters, crabs, worms, mussels, barnacles, echinoderms, sponges, sea anemones, and tunicates. Eighty to ninety different fish species live in the Bay.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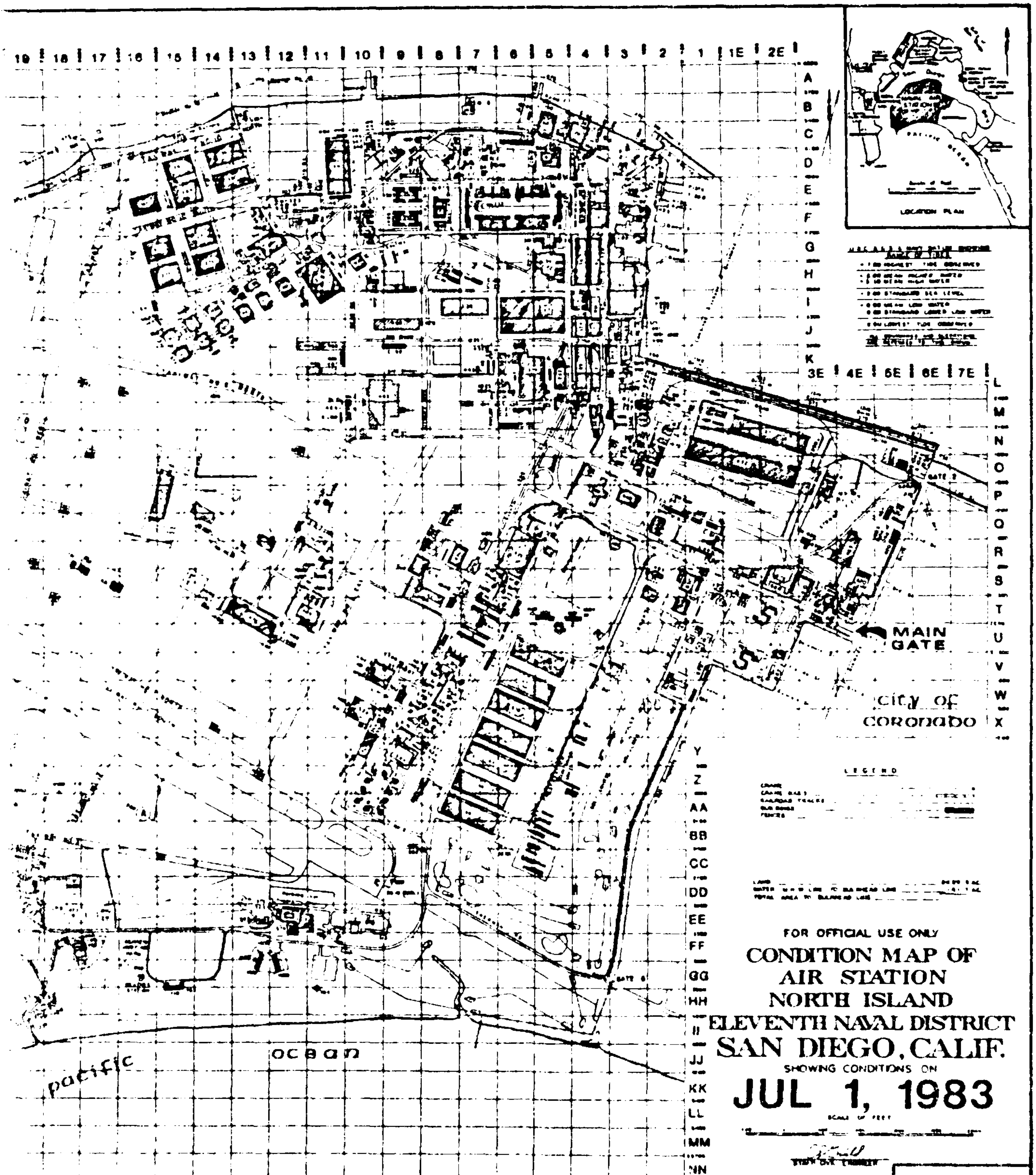


LOCATION PLAN
MAIN GATE
city of coronado
pacific
ocean
FOR OFFICIAL USE ONLY
CONDITION MAP OF
AIR STATION
NORTH ISLAND
ELEVENTH NAVAL DISTRICT
SAN DIEGO, CALIF.
SHOWING CONDITIONS ON
JUL 1, 1983

FIG. 1

SECTION 3 - INSPECTION PROCEDURE

3.1 LEVEL OF INSPECTION

The on-site underwater inspection phase of the work was performed by teams composed of registered engineers with one engineering technician tendering some of the time. All inspections were conducted in the period between July 20 and August 2, 1984.

Photographs were taken by a commercial underwater photographer supported by the engineering team on July 31, 1984.

The inspection techniques were dictated by the requirements of the Scope of Work and the need for that quality of inspection that would yield the proper information to support accurate assessment and recommendation for the structure inspected.

3.2 INSPECTION PROCEDURE

The work was conducted using from two to four engineering divers at any one time with either a diver or a technician as tender. The divers were in the same vicinity at all times so that the single tender did not represent a violation of safe diving standards. Communication between diver and tender was by voice.

A Level I general examination was performed on all Pier piles within each of the open type structures. The Level I examination is essentially a swim-by overview which does not involve cleaning of any structural elements.

The bulkhead Level I examination included an observation of the wall at three levels: mudline, just below MLW, and in the splash zone.

A Level II examination was also performed on 15% of the piles at Pier Bravo. This included hand cleaning of biofouling or debris on three sides or faces of each square pile to an approximate length of 10 inches to expose underlying pile surface at three heights: mean low water, mudline, and halfway between those elevations.

The Level II examination of Pier J/K was limited to the cleaning of approximately 35 piles with a dual objective. It was intended that a sufficient number of piles be inspected to confirm the opinion of pile damage stated in the report of the 1981 inspection of this pier. It was also intended to use various equipment designed for inspection purposes being developed by NCEL, Port Hueneme.

The Level II examination at the bulkheads included cleaning the sheet pile as follows: At the concrete sheet piling every 200 linear feet a 12 inch square area was cleaned at the three elevations described above.

The concrete piling (both bearing piles and sheet piles) were then struck with a pointed hammer at all three elevations to gauge the soundness of the concrete. That soundness was then recorded according to the following nomenclature:

1. Hard: Pick rebounds without making a significant indentation, usually accompanied by a ringing sound clearly heard in the water.

2. Firm: Pick rebounds with a small indentation.

3. Soft: With six blows, 1/4 inch to 1/2 inch indentation can be made.

4. Very Soft: Six blows removes corner of the pile or in excess of 1/2 inch of material.

Record of structural assessment of the concrete sheet piles is shown in Table 5.1.

Chipping was attempted at all four exposed corners at each elevation of all bearing piles and the soundness was recorded.

Each pier pile was inspected at its upper connection to the cap beam for evidence of driving fracture.

It should be noted that non-destructive methods of inspection were used in this project. The conditions noted reflect direct observation coupled with an intimate knowledge of the facilities gained by this office from 25 years of experience with the waterfront structures at the Naval Air Station.

3.3 INSPECTION EQUIPMENT

Equipment used included the usual divers' equipment with scuba gear. Photography equipment included a Nikonos III camera with 15mm wide angle lens and two SR 2000 strobe lights. Chipping hammers and bar scrapers were used to clean and test the piles.
The NCEL equipment included:

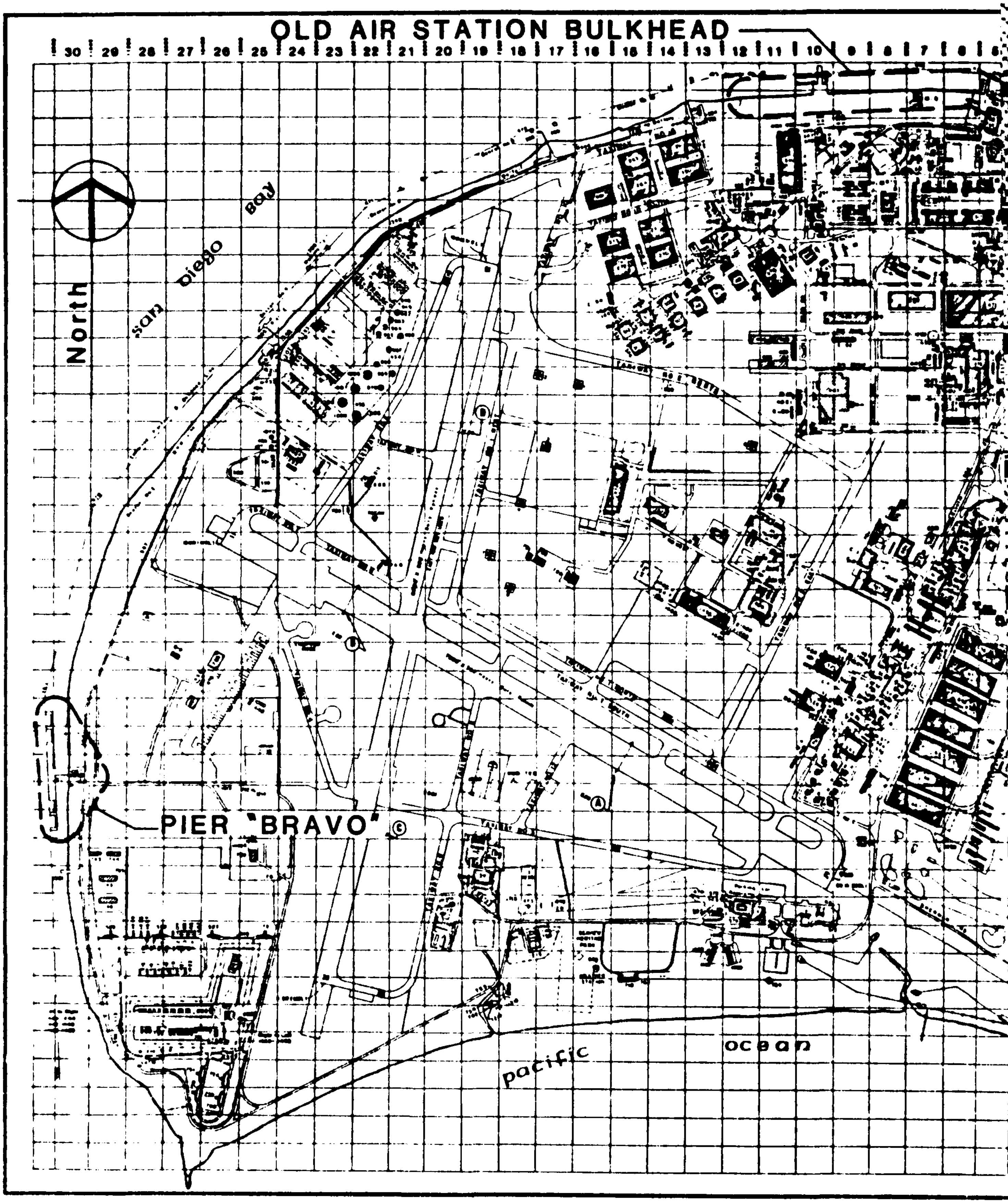
1. A Schmidt hammer modified for underwater use. The hammer measures surface hardness.

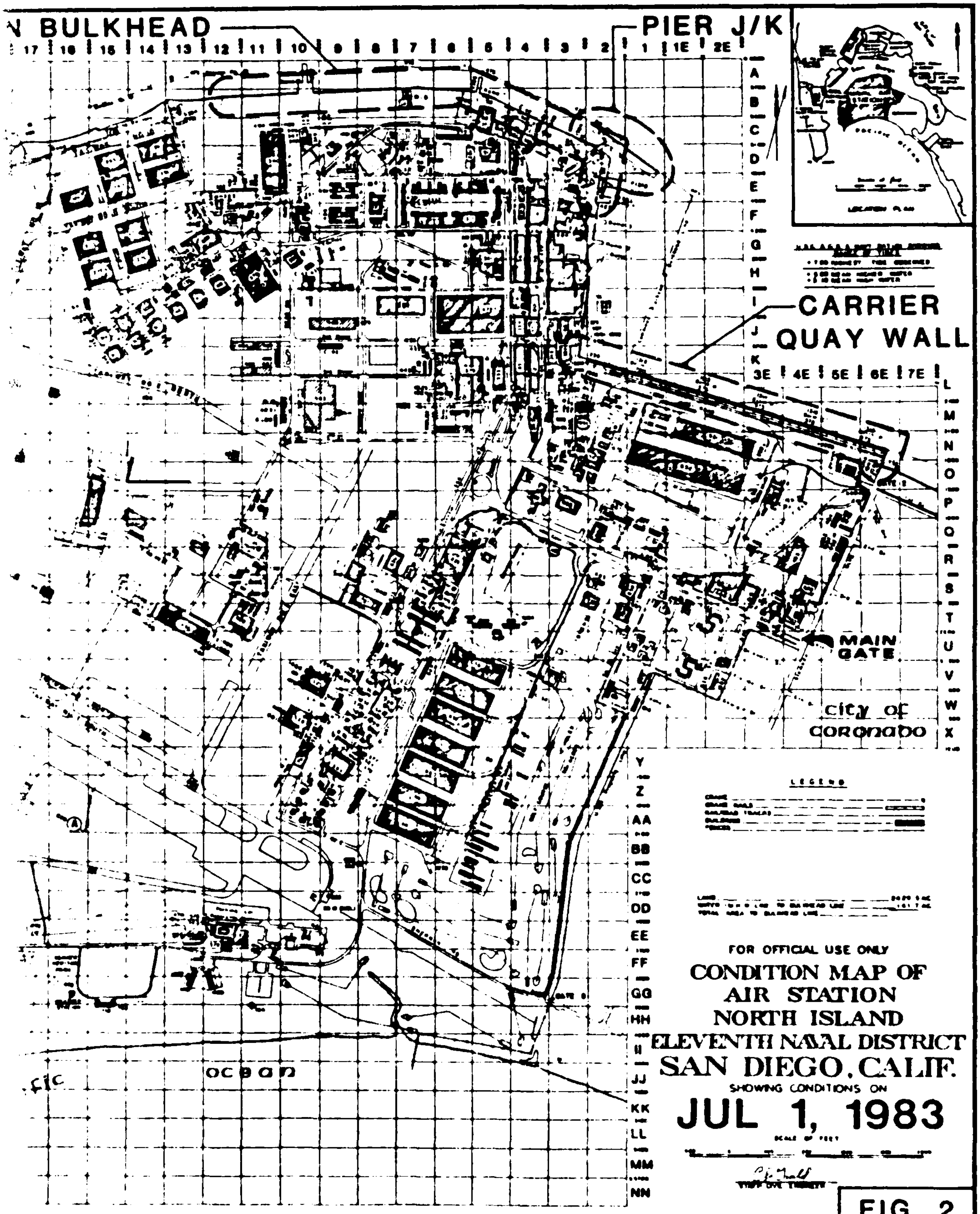
2. A hydrospray device intended for surface cleaning underwater. It is effectively a very high pressure water jet.

3. A hydraulic whirl away rotary abrading tool, for cleaning underwater surfaces.

4. An R meter or pachometer modified for underwater use. The meter is useful in determining the location and size of reinforcing in reinforced concrete.

5. An ultrasonic device used to determine average strength of concrete.


OLD AIR STATION BULKHEAD
North
san diego bay
PIER "BRAVO"
pacific
ocean


N BULKHEAD
PIER J/K
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LEGEND
FOR OFFICIAL USE ONLY
CONDITION MAP OF
AIR STATION
NORTH ISLAND
ELEVENTH NAVAL DISTRICT
SAN DIEGO, CALIF.
SHOWING CONDITIONS ON
JUL 1, 1983
LOCATION PLAN

FIG. 2

SECTION 4 - FACILITIES INSPECTED

4.1 OLD AIR STATION BULKHEAD

4.1.1 DESCRIPTION OF THE FACILITY

The Old Air Station Bulkhead extends from a dogleg south of Building 316 north and west to its seaward end adjacent to the helicopter ramp area, a total distance of 4580 ft.

The bulkhead was constructed in increments and apparently at various times. Drawings for the complete wall are not available. However, the North Island PW files include two single drawings of a tie back wall in the vicinity of Pier J/K.

The older of the drawings dated May 19, 1922 refers to Specification Number 4651 and Y & D Number 95808. It shows the then existing wall to extend from the base of Pier J easterly for about 170 ft. and then southerly another 115 ft. The additions are shown to extend further southerly to the dogleg described above. A west projection is shown to extend about 1050 ft. broken for a distance of 165 ft. by a ramp into the bay and a paved area not presently existing.

The second drawing appears to be a contractors drawing indicating also Contract Number 4651, Ross Construction Co., and dated Jan. 8, 1924. The wall cap elevation on both drawings is shown at EL. +10.00 with no basis of elevations given. The wall is shown as concrete T&G sheet piling with sheets 8" thick by 12" wide.

The base of the piling is shown as EL. -16.5 in pencil on both drawings. A reinforced concrete horizontal wale is shown at EL. +2.5 with concrete encased 1-1/2" round tie rods at 8 feet on centers extending 30' back from the wall to 4 foot square dead-men. The contractors drawing shows the reinforcing steel at the wrong face of the dead man.

4.1.2 OBSERVED CONDITIONS

At the present, the tied wall extends westerly from Pier J/K for approximately 655 feet. The next 60 feet of wall is of gravity cross-section (this is the position of the old ramp since removed). The tied wall continues for 375 feet further to the West. From that location, the wall is gravity type to the western limit, a total distance from Pier J/K of 1078 feet.

The tied back portion of the wall appears to have been gunited and rough troweled above the wale with the joints between piles still showing below the wale.

The mudline elevations vary along the wale as shown in the accompanying drawing. The deepest section is at the dogleg at Building 316. Much of the mudline is above MLLW.

The tied back wall below the horizontal wale exhibits considerable sulphate damage from soft to very soft on the assessment scale. Damage appears to be worst in the vicinity of Pier J/K.

Westward from Pier J/K there is intermittent leakage of the tied-back wall accompanied by occasional surface subsidance behind the wall. This is typical of this type of wall with noncohesive backfill. The soil migrates through the pile joint into the bay helped by the pumping action of the tides resulting in surface cavities.

To the west of Station 16+68 the wall appears to consist of a cast in place gravity retaining wall. The concrete does not exhibit the degree of sulphate damage of the tied-back wall. There is no soil leakage.

At construction joints of the cast in place wall, occasional raveling of the concrete was observed. There are some vertical cracks

midway between construction joints of the wall. They are not considered serious. There are a number of locations where concrete and asphalt rubble have been placed along the face of this wall.

4.1.3 STRUCTURAL CONDITION ASSESSMENT

The structural condition of the cast in place wall west of Station 16+68 is good. The joint raveling, vertical cracking and shallow sulphate damage are not considered to seriously impair the walls ability to support its retaining load.

The condition of the tied back wall east of Station 16+68 varies with location. In the vicinity of the Pier J/K the average depth of sulphate damage is approximately 1 inch, measured under the wale.

This is considered as significant damage for a pile whose total thickness is only 8 inches. Also, this measured loss of thickness is occurring on the compression face of wall where the loss of effective depth represents loss of bending capacity as a squared function ($d^2 = M/K$). A 1 inch loss of effective depth in this wall represents a loss of about 30% of the moment carrying capacity of the cross section.

For design loads as a simple retaining structure without significant surcharge the wall is capable of supporting its backfill. However, surcharge in excess of 250 pounds per square foot would be expected to overload the wall where its greatest sulphate damage has been sustained.

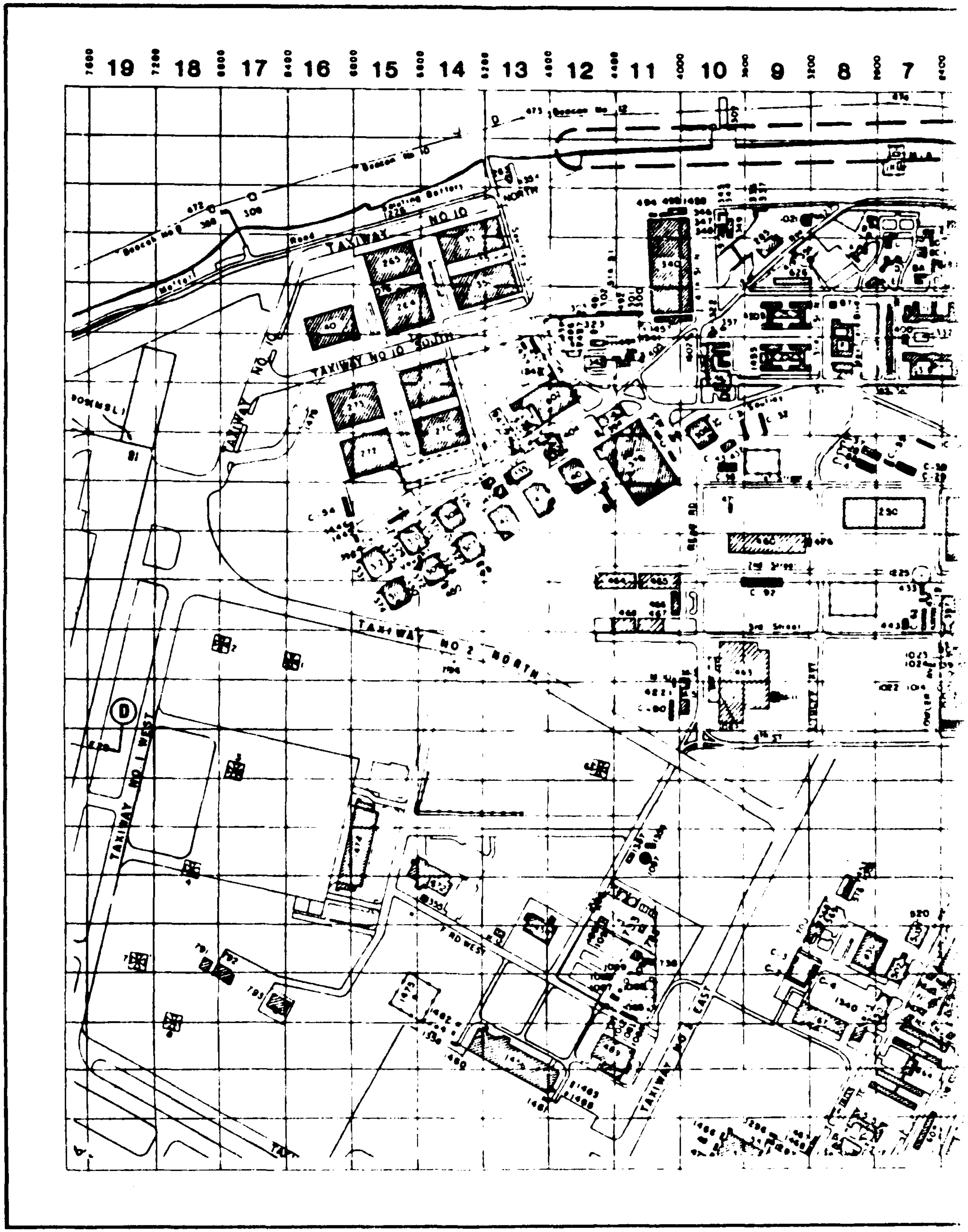
4.1.4 RECOMMENDATIONS

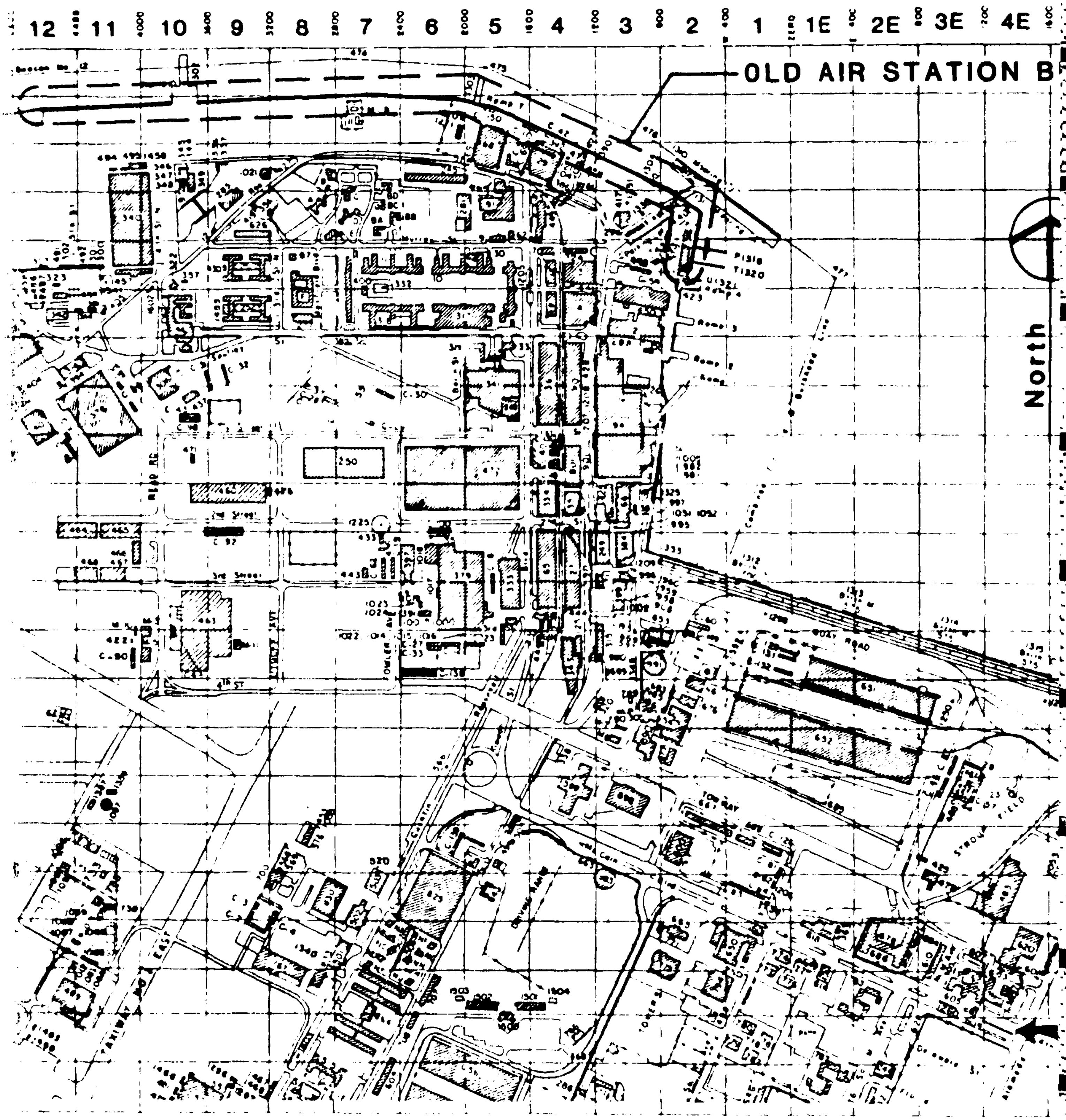
The tied back wall east of Station 16+68 wall has sustained sulphate damage which has reduced its ability to sustain the loads for which

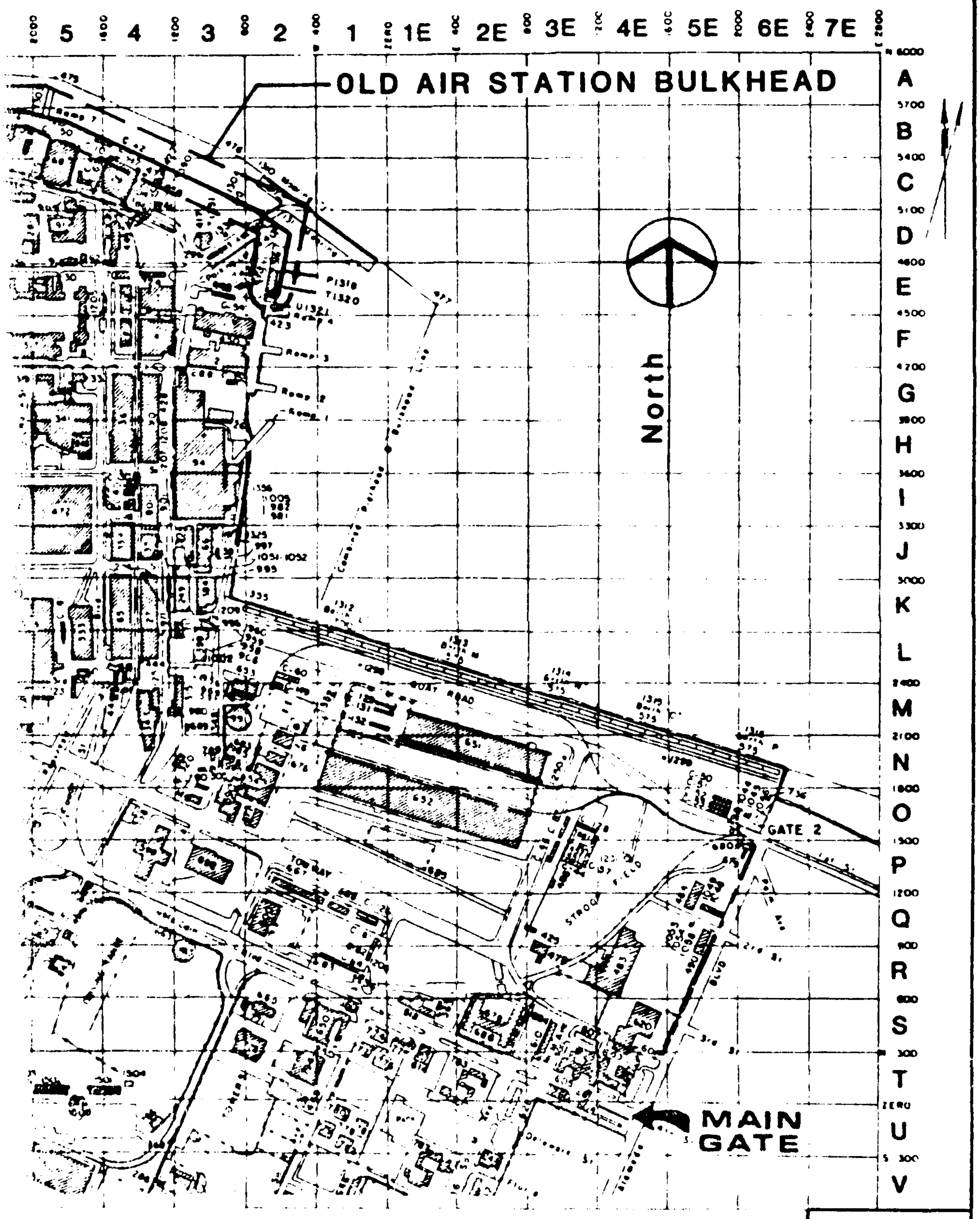
it was designed. But there is still sufficient strength in the cross section remaining to support normal loads.

It is recommended that the wall be strengthened or replaced if surcharges in excess of 200 pounds per square foot are planned in areas adjacent to the wall.

At various locations the tied wall has sustained leakage of backfill into the bay. The surface evidence of this leakage are surface cavities which have occurred immediately to the rear of the wall. These cavities represent a hazard to traffic both by foot and vehicle. It is recommended that their filling be a regular maintenance function, and the wall inspected again in five years.

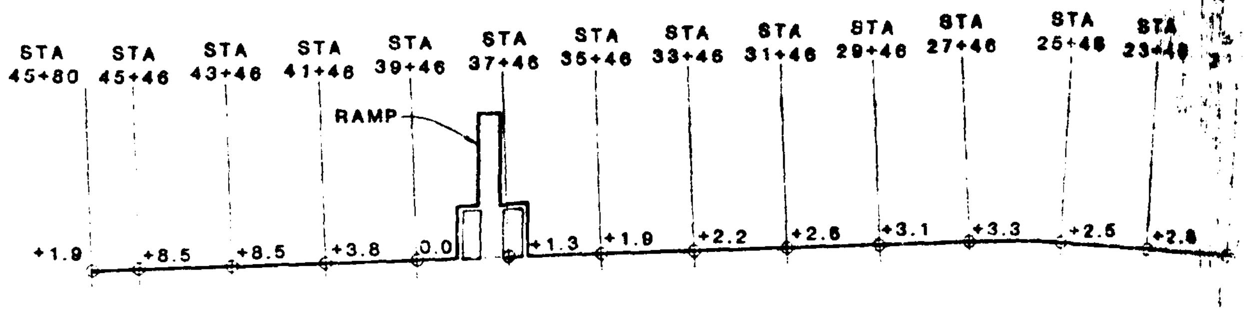

19
18
17
16
15
14
13
12
11
10
9
8
7
TAXIWAY NO 10
TAXIWAY NO 10 SOUTH
TAXIWAY NO 2 NORTH
TAXIWAY NO 1 WEST
TAXIWAY NO 2 EAST
D


12
11
10
9
8
7
6
5
4
3
2
1
1E
2E
3E
4E
OLD AIR STATION B
North
250
2nd Street
3rd Street
4th ST
P1318
T1320
QUAY ROAD



5
4
3
2
1
1E
2E
3E
4E
5E
6E
7E
OLD AIR STATION BULKHEAD
North
MAIN GATE
GATE 2
A
B
C
D
E
F
G
H
I
J
K
L
M
N
O
P
Q
R
S
T
U
V

FIG. 3



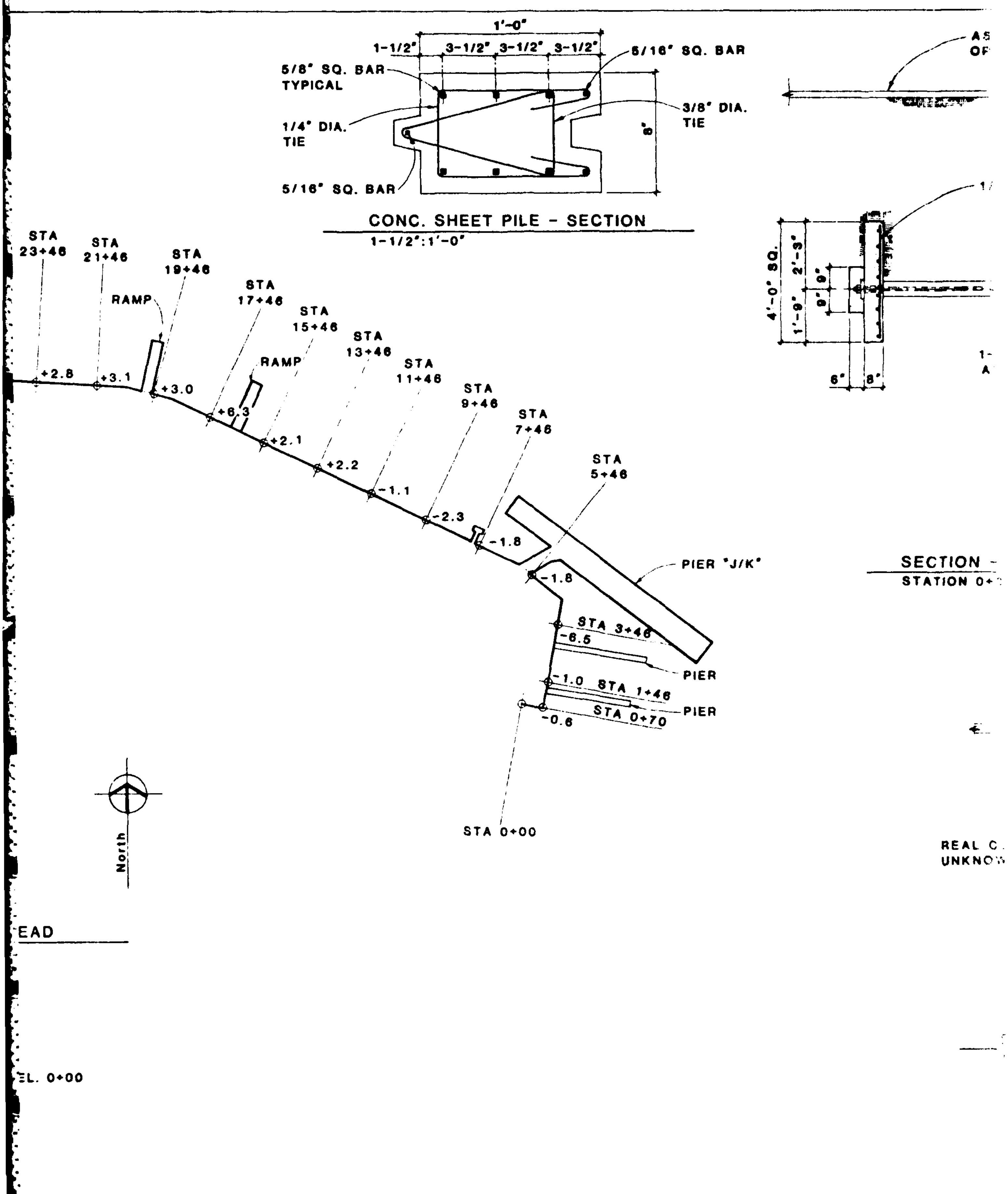
KEY PLAN - OLD AIR STATION BULKHEAD

1":400'

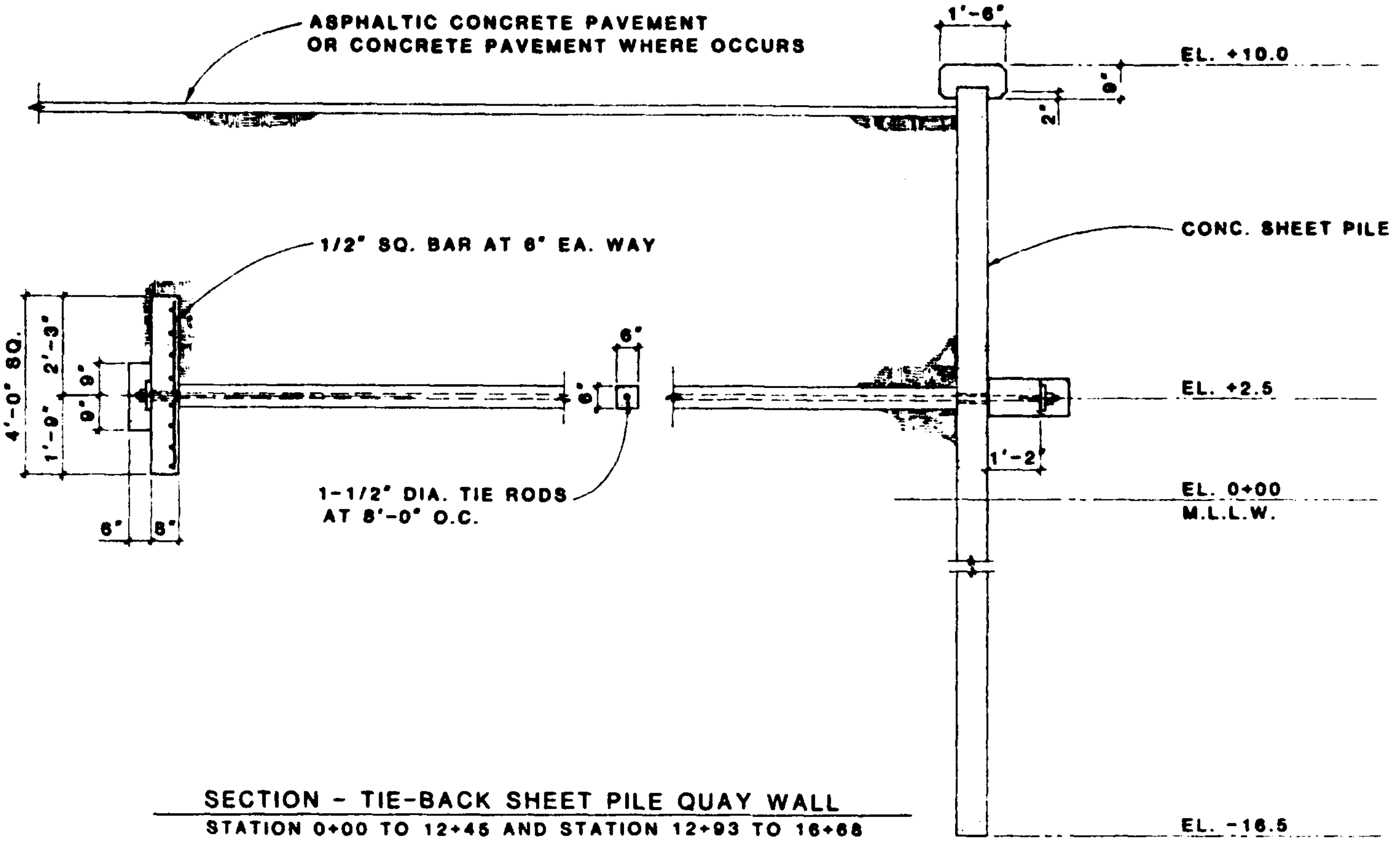
NOTES:

1. ○ INDICATES CLEANED STATION

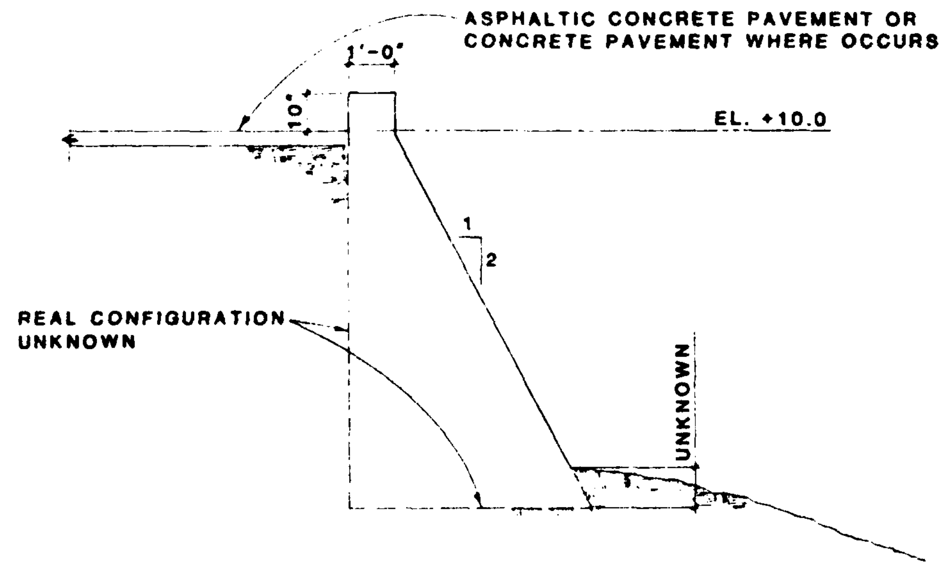
2. +1.9 INDICATES MUDLINE ELEVATION
MEAN LOWER LOW WATER DATUM - EL. 0+00



1'-0"
1-1/2"
3-1/2" 3-1/2" 3-1/2"
5/16" SQ. BAR
5/8" SQ. BAR TYPICAL
1/4" DIA. TIE
3/8" DIA. TIE
8"
5/16" SQ. BAR
CONC. SHEET PILE - SECTION
1-1/2":1'-0"
STA 23+46
STA 21+46
STA 19+46
RAMP
STA 17+46
STA 15+46
RAMP
STA 13+46
STA 11+46
STA 9+46
STA 7+46
STA 5+46
+2.8
+3.1
+3.0
+6.3
+2.1
+2.2
-1.1
-2.3
-1.8
-1.8
PIER "J/K"
STA 3+46
-6.5
PIER
-1.0
STA 1+46
PIER
STA 0+70
-0.6
STA 0+00
North
4'-0" SQ.
2'-3"
1'-9"
9"
9"
6"
8"
SECTION -
STATION 0+
REAL C.
UNKNOW
EAD
EL. 0+00

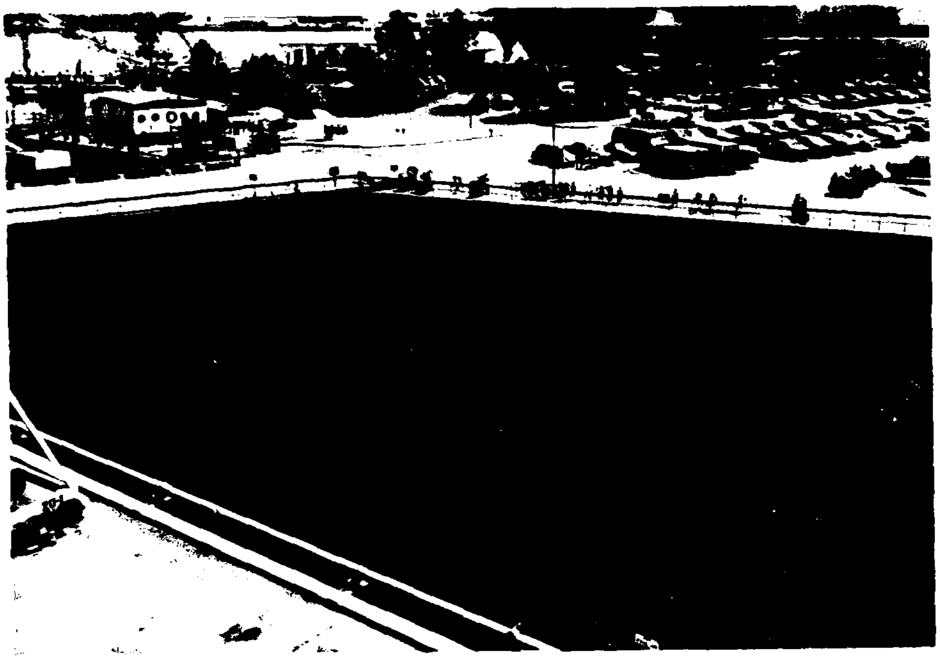


SECTION - TIE-BACK SHEET PILE QUAY WALL
STATION 0+00 TO 12+45 AND STATION 12+93 TO 16+68

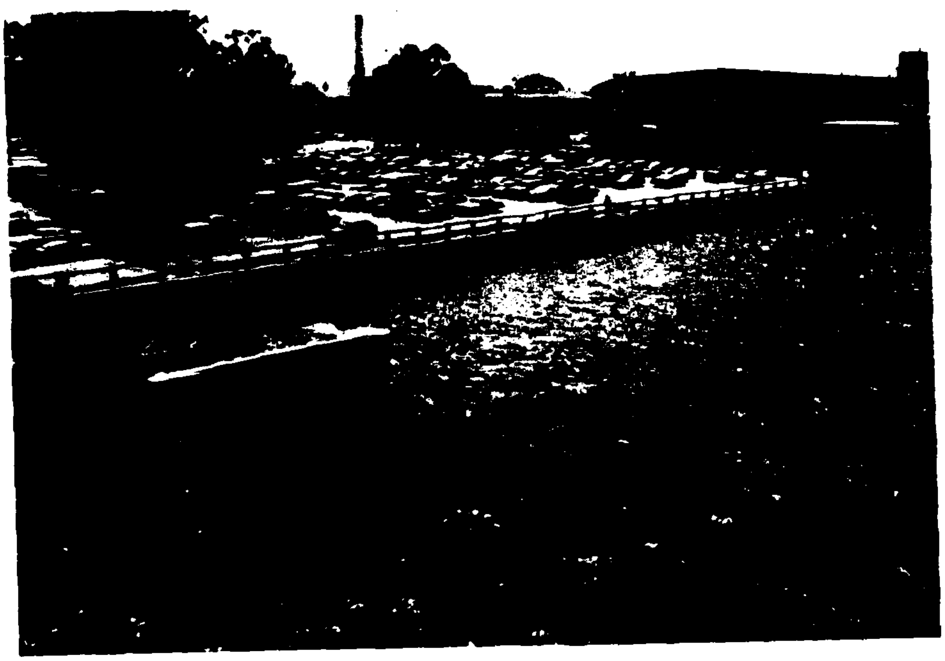


SECTION - TYPICAL - GRAVITY WALL
1/4":1'-0"

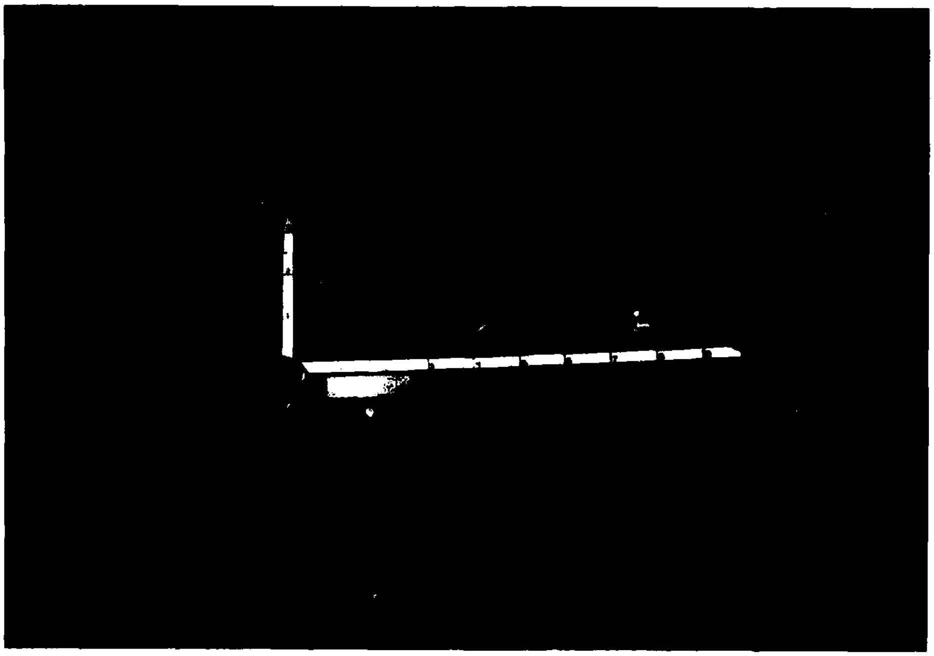
OLD AIR STATION BULKHEAD PLAN AND TYPICAL SECTIONS NAVAL AIR STATION - NORTH ISLAND, SAN DIEGO, CALIFORNIA		
Blaylock-Willis and Associates STRUCTURAL ENGINEERS SAN DIEGO, CALIFORNIA	DATE: OCT. 1984	FIG. 4



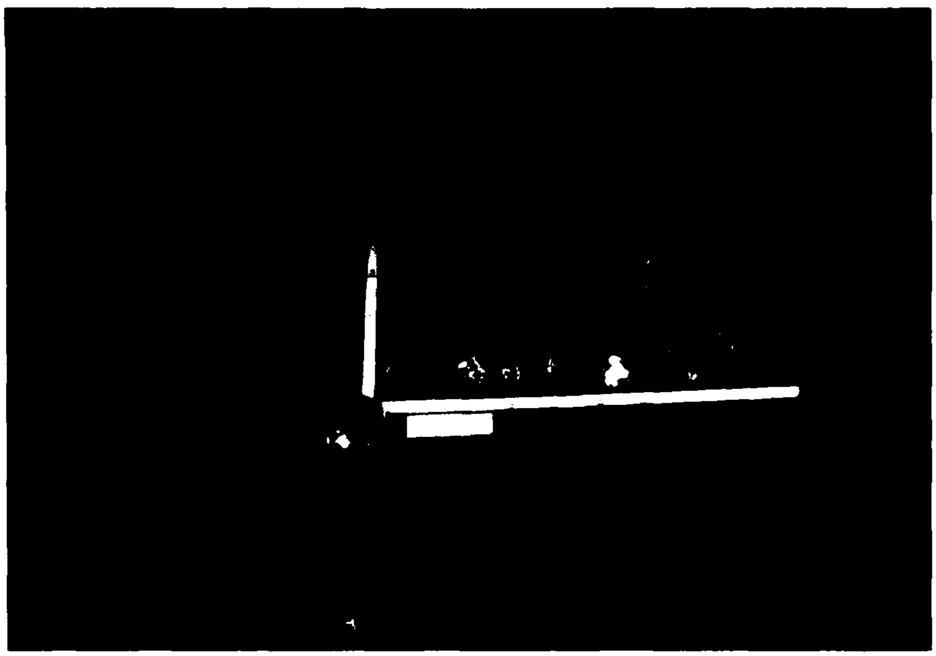
1. Old Air Station Bulkhead at Pier J/K. Picture from derrick platform looking South.



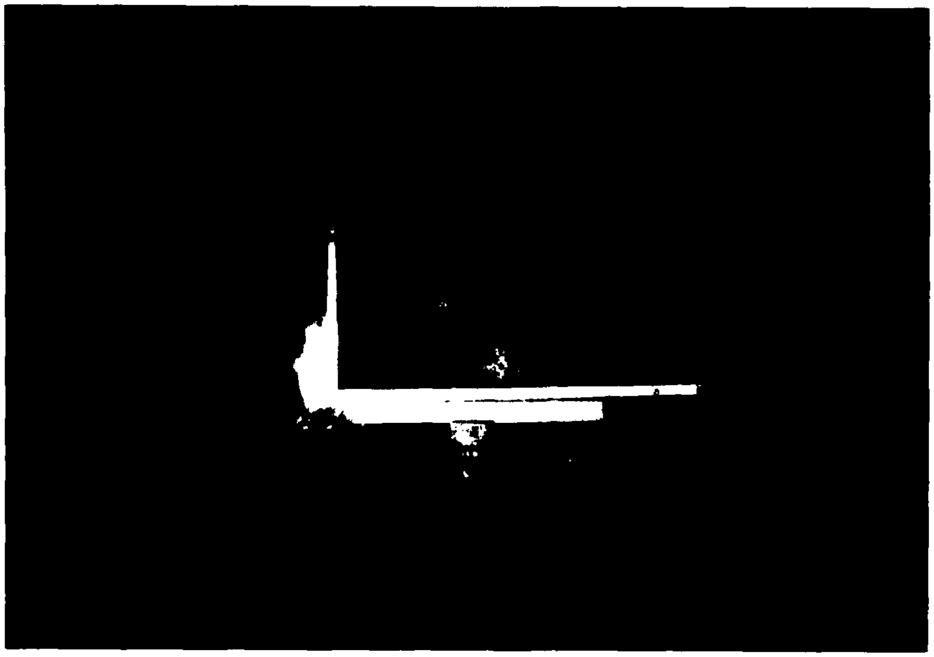
2. Bulkhead West of Pier J/K



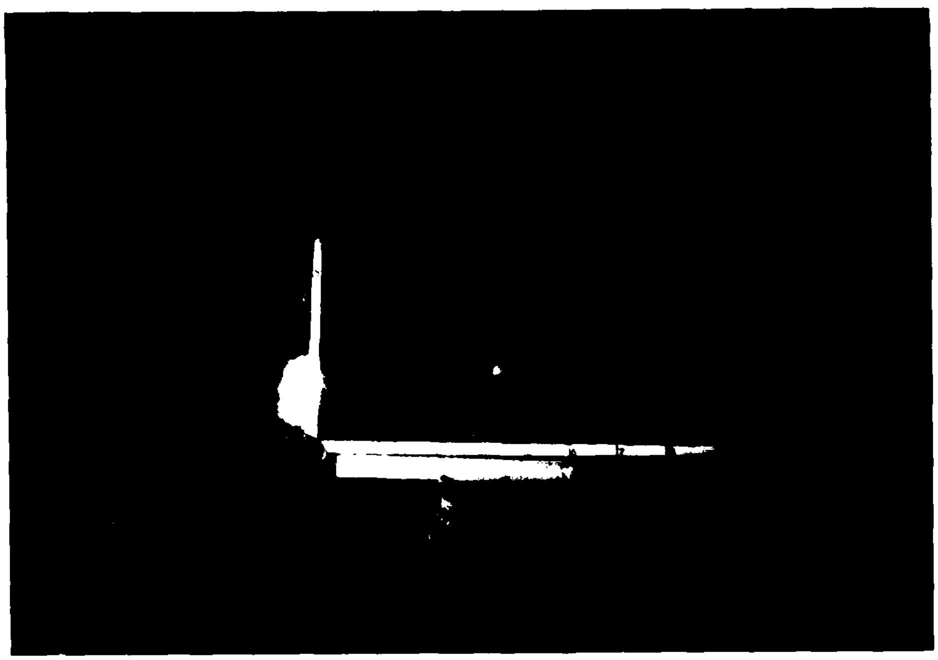
3. Bulkhead, cleaned area at Station 5+46 just above mudline previous to picking with pointed hammer.



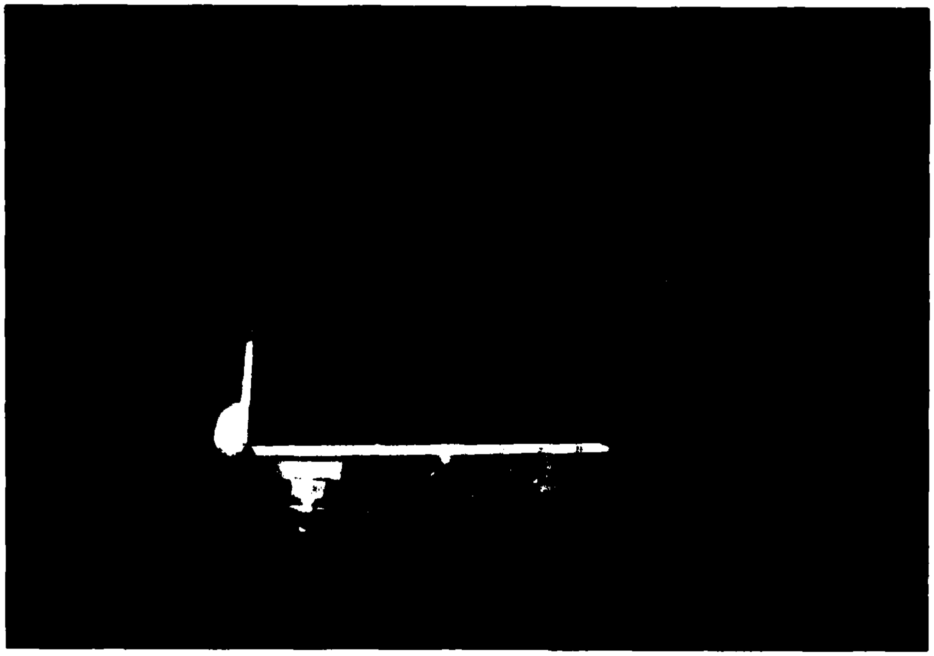
4. Bulkhead, at Station 5+46. Cleaned area after half dozen blows of pointed hammer has made crater 1½ inches deep. Sulphate attack has softened surface concrete.



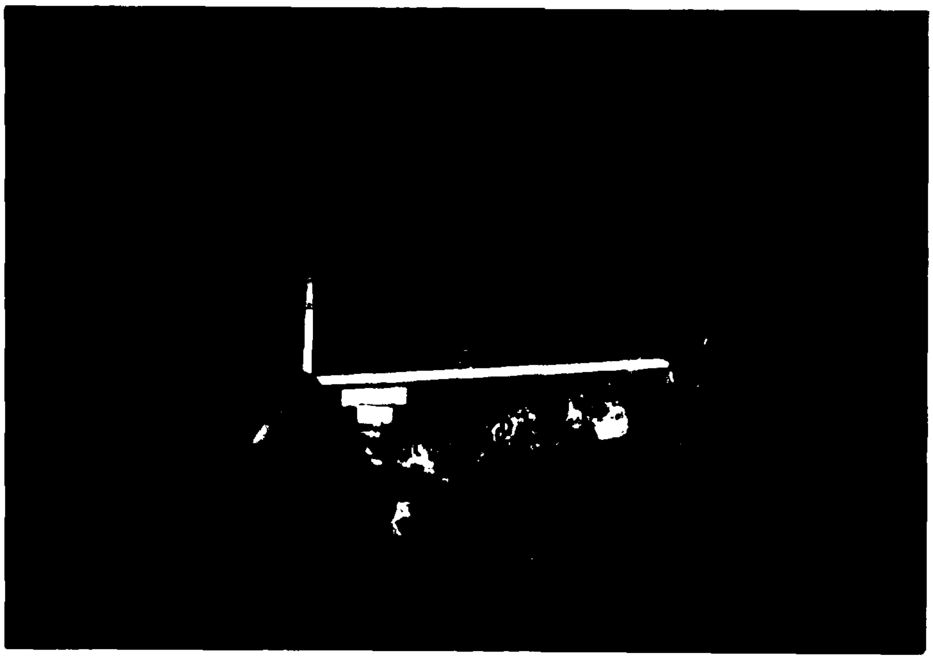
5. Bulkhead, Cleaned area just North or West of stem of Pier J/K. View is of concrete sheet pile joint below the wale.



6. Bulkhead, location is the same as Photo 5. Corner of pile has been removed by six blows of pointed hammer. Concrete is soft as the result of sulphate attack.



7. Bulkhead, cleaned area below wale at Station 7+46.



8. Bulkhead, cleaned area below wale at Station 7+46. Six blows of the hammer have eroded a small spall 3/4 inch in depth adjacent to 4 on horizontal scale. Sulphate damage is not so pronounced as nearer Pier J/K.

4.2 CARRIER BULKHEAD

4.2.1 DESCRIPTION OF THE FACILITY

The Carrier Quay Wall or Bulkhead at Spanish Bight was constructed in 1945 from plans prepared by the Eleventh Naval District (See P.W. Drawings No. NA11/N15-3, Specification No. 13977). Plans were entitled, "Quay Wall at Spanish Bight, Change Order Accompanying NOy9445". The wall is approximately 3,325 feet in total length with a 200 foot return at the Coronado or east end. It consists of reinforced concrete tongue and groove sheet piling supported by a relieving platform. Originally designed for a timber platform, it was constructed finally of reinforced concrete of thickness varying between 3 feet and 2 feet 6 inches. The relieving platform is supported by untreated timber piling. Elevation of the concrete sheet piling at deepest penetration is -60.0 feet MLLW. The design bottom of bay adjacent to the wall is -35.0 feet and -40.0 feet. The original mudline in this area was approximately -20.0 feet. The present mudline at the face of the wall varies from -27.0 feet to -40.0 feet in the deeper areas. Timber piles were driven to various depths, the deepest penetration being to -54.0 feet (see typical section through the wall). Above the relieving platform, a poured-in-place reinforced concrete cap forms a retaining wall for the earth backfill. Top of wall is at EL. + 11.5 MLLW and bottom of relieving platform at EL. + 1.5.

The wall originally was constructed in the "dry." A berm was constructed outboard of the location of the sheet piling. A well point system was used to lower the elevation of ground water so that activities at the elevation of the relieving platforms could continue at tide elevations above EL. + 1.5. The concrete sheet piling was placed almost entirely blind from the working surface to final elevation (Maximum -60). The design configuration of the tongue and groove detail was such that the

tongue did not entirely fill the groove; the space being left for final grouting with the expectation of sealing the joint in this manner. However, intrusion of cohesive soils into some of the void spaces occurred and the grouting operations, rather than displacing all of the soil and sealing the joint, often produced only icicles of concrete grout surrounded by clay and unbonded to the walls of the grout space. Also, considering the pile configuration and the depth of penetration, it is not surprising that there was some twisting and spreading of the piles during driving. Subsequent dredging outboard of the wall was accompanied by a series of cave-ins, occurring after the outside material was removed. An article appearing in the Proceedings of the American Society of Civil Engineers, January, 1948, entitled, "Cave-Ins of Sandy Backfills" describes the sequence of losses of backfill shortly after the wall was constructed. See Section 5.4. Cave-in No. 4 occurred in the vicinity of the access hole near Sta. 17+00.

After the cave-ins, the pattern of dredging was altered and efforts were made to hand seal the wall from the outboard side. To replace the lost backfill, the contractor pumped material under the relieving platform, using mudjack equipment.

Apparently there have been later efforts made to hand seal still leaking joints by both the contractor and Station personnel; however, detailed records of these efforts are no longer available.

In 1967, the engineering firm A.J. Blaylock and Associates was retained by the Southwest Division, NAVFAC, under Contract No. N62473-67-C-0502 to conduct an investigation of the Quay Wall. The resulting report entitled "Engineering Investigation and Analysis of Subsidence at the Quay Wall, Naval Air Station, North Island" discussed the continuing leakage of noncohesive soils through the joints of the sheet pile, the formation of large voids under the platform and the exposure to the water borne borers, toredos and limnoria.

In 1970, A.J. Blaylock and Associates were retained by Southwest Division to determine an appropriate repair method for the problems associated with the wall, and prepare construction drawings entitled, "Correct Subsidence at the Quay (R26-69)".

The subsequent construction contract was numbered N62473-70-C-0053.

The 1970 repair included the following tasks.

1. The drilling of 2 rows of 4" holes through the relieving platform at 50 feet on centers.

2. The filling of the voids under the relieving platform with a silty sand water mix injected through the 4" holes.

3. The drilling of 2" holes through the joints of the concrete sheet piling.

4. Grouting through the 2" holes at the face of the wall in an attempt to seal the rear of wall against further leakage.

5. The final grouting of the void areas through the 4" holes.

It was estimated that 3000 cubic yards of silty sand and grout material would be needed.

In 1967, three rectangular access holes were drilled through the relieving platform in the utility tunnel. Ten more holes were drilled in 1970. The location of these holes was indicated on the repair project drawings (See Sheet 134551). The object of the access holes was to provide admission to the void areas under the platform where the depth of void would admit access.

In 1967 and 1970, admission was gained into the voids and pictures were taken of the void areas and the damaged piling (See enclosed photos 28 thru 31).

Inspection of the untreated timber piles revealed limnoria and toredo damage. The diameters of the piles appeared to vary considerably. It is estimated that this variation of butt diameter ranges from approximately 12 inches to 16 inches. This is understandable when considering the time and conditions under which the piles were procured.

Damage appeared to vary somewhat from pile to pile. In general, surface damage to the exposed timber approximated 2 inches on the diameter. This was evidenced by protruding knots. A ring of primarily limnoria activity at the intersection of the timber piles with the concrete relieving platform, extending inward from the surface of intersection and measuring a maximum of 3 inches in vertical dimension and a maximum of 3 inches in horizontal dimension, was observed on approximately 60 percent of the pilings. This ring was most pronounced at the outermost row of timber pilings. One piling, estimated at original diameter of 13 inches, was reduced to approximately 6 inches at the base of the ring. Efforts were made to determine the condition of the pilings below the surface of the sand in the void by excavating the soil adjacent to several pilings to the extent of a few inches and feeling the surface. This surface below the void was found to be sound and free from surface borer activity.

4.2.2 OBSERVED CONDITIONS

The present inspection of the Carrier Quay Wall comprised two separate activities, an inspection of the exposed outer face of the concrete sheet piling and an inspection conducted through the relieving platform. The latter inspection involved making soundings

through 2" standpipes located in the floor of the utility tunnel (the relieving platform) and attempts to gain physical access to the underside of the relieving platform through the access holes drilled in 1967 and 1970 and subsequently filled with sand and concrete.

The 1967 and 1970 experiences included the discovery of large voids under the relieving platform and large cones of leaked noncohesive backfill at the mudline adjacent to the face of the wall.

The present inspection found evidence of leakage at the mudline but no single leak which could be called large. Attention directed to evidence of sulphate damage of the sheet piling found only slight damage. Of the nineteen locations cleaned at three elevations, only four locations and six elevations revealed sulphate damage in excess of 1/4 inch. This is not considered significant (See "Discussion of Pertinent Chemical Damage", Section 5.2).

The inspections conducted from the utility tunnel included sounding the 2" standpipes which had been constructed for this purpose. They were cast in place - extending through the total thickness of the relieving platform. Unfortunately, a significant number of them had had their caps removed and were plugged with wooden wedges which station forces were unable to remove. Many others were plugged below the elevation of tunnel floor apparently by grout from the 1970 repair.

It would be helpful to the designing A & E who eventually produces working documents for a current repair of the wall if the wooden plugs and grout intrusions could be drilled out. This would allow a more exact assessment of the quantity of fill material needed to seal the voids.

Between Stations 0+94 and 32+96, sixteen standpipes were sounded. Of these, ten revealed voids which measured between 1 and 13 inches in depth below the relieving platform.

On two occasions, attempts were made to gain physical access to the voids under the relieving platform as had been done in 1967 and 1970. The first attempt was made in the early morning of July 27, 1984 when the tide was below the +1.5 elevation of the bottom of the relieving platform.

Using station maintenance crew, the steel covers over access holes near Stations 17+00, 27+51, 29+93 and 31+75 were taken up. The cover at Station 26+50 could not be removed. The hole near Station 17+00 was found to be filled to a depth of 14" with a quick hardening cement which was removed by jackhammer revealing a wood framework below and sand fill. Evidence of a void was indicated below the wood. The tidal window closed out further attempt at this opening on this date.

The access hole near Station 27+51 was filled with grout and sand which there was not time to remove. The hole near Station 29+93 was filled with sand to the elevation of the bottom of the relieving platform. The hole near Station 31+75 was also filled with sand. No satisfactory opening was found to provide human access to a void or for taking pictures of a void.

Subsequent discussions with the Public Works Office led to a second attempt at gaining access below the platform. A tidal window was presented in the early hours of August 25 midnight to 0600. This time, it was possible to remove the sand from the holes near Station 31+75 and Station 17+00 to the extent that the photographer could get pictures of the voids in the close vicinity of the access holes.

The hole near Station 31+75 revealed a void approximately 6 inches in depth widening somewhat toward the water - as with a sloping beach. However, the void had no regularity and grout could be seen (with the appropriate use of a flashlight and mirror) adhering to the bottom surface of the relieving platform. A pile located a short distance in front of the access hole showed clean surface undamaged by borers. This fact is puzzling as water obviously covers the pile in higher tides and every other pile observed under the platform had surface damage. It could be reasoned that there is not direct access of the tidal water to the pile but rather the water filters through a sand barrier which excludes the borers.

The void near Station 17+00 was large enough so that the inspecting engineer could sit down in the sand at the access hole and extend his legs under the platform but the void was not deep enough at the hole to permit entrance or direct visual inspection. The mirror and flashlight were again used for this purpose. As at Station 31+75, grout could be seen adhering to the bottom of the platform. Piles were visible and the void deepened in the direction of the water. Surface damage of these piles was clearly indicated.

An attempt was made to remove the concrete plug at Station 3+15 but the tidal window ran out before it could be accomplished.

4.2.3 STRUCTURAL CONDITION ASSESSMENT

The original construction drawings (See P.W. Drawing No. NA11/N15-/3(20)) show the Quay Wall to have been designed for the following loads.

A. Design Assumptions for Sheet Piles and Retaining Wall Per Foot of Wall

Live Load Surcharge	=	600 #/S.F.
Bitt Load	=	1300 #
W = Weight of Sand	=	90 #/Cu.Ft.
W_1 = Weight of Seawater	=	64 #/Cu.Ft.
W_2 = Weight of Submerged Fill	=	60 #/Cu.Ft.
W_3 = Weight of Undredged Bottom	=	60 #/Cu.Ft.
W_4 = Equivalent Liquid Pressure of Fill		
$= W \tan^2(45^\circ - 1/2\ \emptyset)$		
$= 90 \times \tan^2 27^\circ$	=	25 #/Foot
W_5 = Equivalent Liquid Pressure of Saturated Fill		
$= W_1 + W_2 \tan^2(45^\circ - 1/2\ \emptyset') = 64\# + 60\# \times \tan^2 31^\circ 43'$	=	87 #/Foot
W_6 = Equivalent Liquid Pressure of Active Earth Pressure		
$= W_1 + W_3 \tan^2(45^\circ - 1/2\ \emptyset'')$		
$= 64\# + 60\# \times \tan^2 25^\circ$	=	77 #/Foot
W_7 = Equivalent Liquid Pressure of Passive Earth Pressure		
$= W_1 + W_3 \tan^2(45^\circ + 1/2\ \emptyset'')$		
$= 64\# + 60\# \times \tan^2 65^\circ$	=	340 #/Foot
$\emptyset$ = Angle of Repose of Fill	=	$36^\circ 00'$
$\emptyset'$ = Angle of Repose of Saturated Fill		$26^\circ 34'$
$\emptyset''$ = Angle of Repose of Undredged Bottom	=	$40^\circ 00'$

Except at the utility tunnel, the structural condition of the Quay Wall can be described as good.

The utility tunnel is exhibiting some structural deterioration. Particularly in the 8" ceiling slab, the reinforcement has rusted to the extent that concrete cover has spalled off exposing the reinforcing steel. The problem is clearly indicated at all of the manholes (See photographs). Obviously, the condition serves to reduce the capacity of the slab to support loads.

Also, the utility tunnel is in need of a good housecleaning. Material of various sorts have accumulated throughout the tunnel.

The slight surface sulphate deterioration of the concrete piling is not serious. However, the continuing formation of voids under the relieving platform is considered very serious and the prevention of further soil leakage into the bay must be addressed.

4.2.4 ALTERNATIVE REPAIR SOLUTIONS

In evaluating the Repair Solutions, the following requirements are considered.

1. The repair must insure as much as possible against the free flow of saltwater to the wooden piling supporting the relieving platform in order to prevent further marine organism damage to the piling.

2. The repair must not result in damage or the threat of damage to the structural elements comprising the pier.

3. The repair must not contain material or ingredients which can harm the ecology of the area.

The repair methods considered are as follows.

<u>Method 1</u>. Modified 1970 Solution. Seal the wall from the exterior face and grout all voids full.

<u>Method 2</u>. Hand seal the leaking joints and grout voids full.

<u>Method 3</u>. Place a steel sheet pile form over the entire face of wall. Grout between walls and grout voids full.

Specific consideration of each method is as follows.

<u>Method 1</u>

It is intended to seal the wall on its interior face and fill the existing voids. The construction sequence is as follows:

a. Core 2 inch holes at 8 feet vertical centers at each fourth concrete sheet pile joint and at obviously leaking joints.

b. Pump cement grout under pressure through each hole starting at lowest hole.

c. Grout voids through row of holes cored 15' from the landside edge of the relieving platform.

Some loss of grout through the joints of the bulkhead is to be expected and will necessitate packing of the joint with paper or rags as the grouting progresses. This leaking has the advantage of giving evidence of the disposition of the grout.

This method has the following advantages:

a. It is a repair method used before in the San Diego area and indeed at the Quay Wall.

b. The costs of the major steps in the construction sequence can be readily determined.

This method has the following disadvantages:

a. The success of the method depends upon the integrity of the personnel accomplishing the work. While it is possible to observe the underwater portion of the construction periodically, it is difficult or impossible to monitor at all times.

b. The presence of 1970 grout in the vicinity of the new grout holes could prevent the effective placement of the grout behind the wall. In this regard, we are not starting cleanly as was the case in 1970.

c. Care must be exercised during grouting of the face of the wall to prevent excessive lateral pressures from developing which would endanger the stability of the Wall. While it is possible to monitor these pressures, it is difficult to control them without constant attention.

d. The existing mudline varies along the wall but in the deeper sections is approximately -30 feet (MLLW). Leakage of wall backfill material is through concrete pile joints exposed above the mudline. If future dredging

results in the lowering of the mudline at the wall and exposing lower wall area now covered by soil, further efforts must be made to seal these exposed areas.

It is estimated that the total cost of this method is the sum of $1,209,500 (See Section 5 for a detailed cost estimate).

<u>Method 2</u>

It is intended by this method to fill the voids under the relieving platform from the surface through 2 row of holes cored through the relieving platform. The leaking joints between sheet piling are to be cleaned and hand filled with an appropriate sealing material.

The construction sequence is as follows:

a. Drill two rows of holes parallel with the wall through the relieving platform. One row located at the utility tunnel and the other 15 ft. from the landside edge of the relieving platform.

b. Grout voids full through surface holes.

c. Clean designated pile joints of obstructing materials, loose grout and marine growth by hand scraping and sandblasting.

d. Pack sealing material into cleaned joints.

This method has the following advantages:

a. It allows the joints which are obviously leaking to be selected (at the design phase) and treated separately.

b. It provides for filling the voids from the surface where the activity can be more easily monitored.

c. The results of the hand sealing operation can be observed and inspected.

This method has the following disadvantages:

a. There is sometimes difficulty in determining whether or not a joint is leaking - particularly the narrower ones.

b. Difficulty can be expected in cleaning and sealing the narrow leaking joints.

c. The larger joints which already exhibit hand sealing efforts of the past are messy and difficulty would be expected in getting new material in and around the old materials.

d. The success of the method depends upon the integrity of the contractor and the ability of the Navy to inspect the work as it proceeds.

e. Future dredging must consider exposure of uncovered wall area susceptable to increase soil leakage.

It is estimated that the total cost of this method is the sum of $695,700 (See Section 5 for a detailed cost estimate).

<u>Method 3</u>

It is intended by this method to erect a steel sheet pile form

outboard of the present concrete wall. The new form would be connected at the top to a poured-in-place concrete connection to the cap. Lean concrete would then be placed in the void between the two walls. The voids under the relieving platform would be filled through holes drilled from the surface as with Method 2.

The construction sequence is as follows:

a. Drill holes through the relieving platform in two rows as with Method 2.

b. Grout voids below the relieving platform through the surface holes.

c. Remove existing buttresses from the face of the Quaywall.

d. Drive PZ 27 sheet piling.

e. Dowel, reinforce and cast concrete cap extension on the face of wall.

f. Fill the space between the concrete sheet piling with a concrete fill material.

The advantages of this system are as follows:

a. It is the surest method of solving the leakage problem characteristic to concrete sheet pile bulkheads.

b. It guarantees the greatest protection to the untreated wooden piling supporting the relieving platform and assures the greatest service life to the facility predicated on the protection of the wood piling.

c. Its construction can be monitored by usual inspection methods.

d. The problem of increased leakage due to the exposure of the lower face of the Quay Wall described as a disadvantage in the previous methods does not apply. The steel sheet piling can be driven deeply enough to preclude exposure of the concrete pile face.

The disadvantages of the system are:

a. It is the most expensive of the methods considered.

b. Optimum service of the sheet metal wall needs the provision of a maintained cathodic protection system.

c. Its installation will interfere with normal operations of the Quay more so than the other systems considered.

It is estimated that the total cost of this method is the sum of $5,408,000 (See Section 5 for a detailed cost estimate).

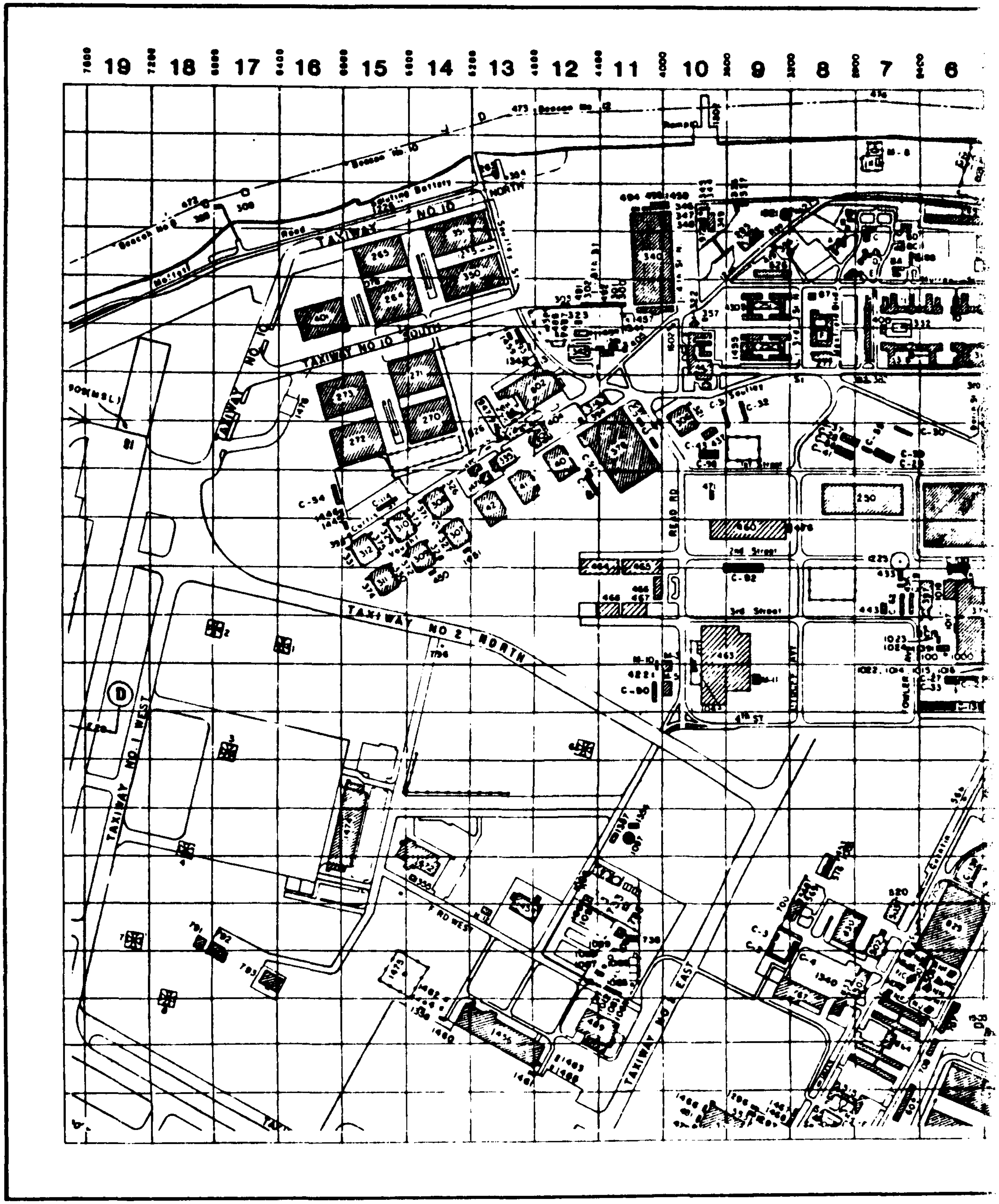
4.2.5 RECOMMENDATIONS

As described above, the principal objectives of the suggested repairs is to seal the wall against the migration of backfill material into bay.

Method 1 described above is very similar to the 1970 repair. While the repair quite obviously reduced the rate of soil leakage, it did not completely stop the leakage. There is question as to its ability to stop the leakage if tried again. There is also the problem of future lower wall leakage.

Method 2 would appear to rely very heavily on the selection and successful filling of small joints. Future leakage of the lower wall is also a problem.

Method 3 although the most expensive has the greatest chance of complete success of all methods. This method is recommended.



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TAXIWAY NO 1 WEST
TAXIWAY NO 4 EAST
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1st Street
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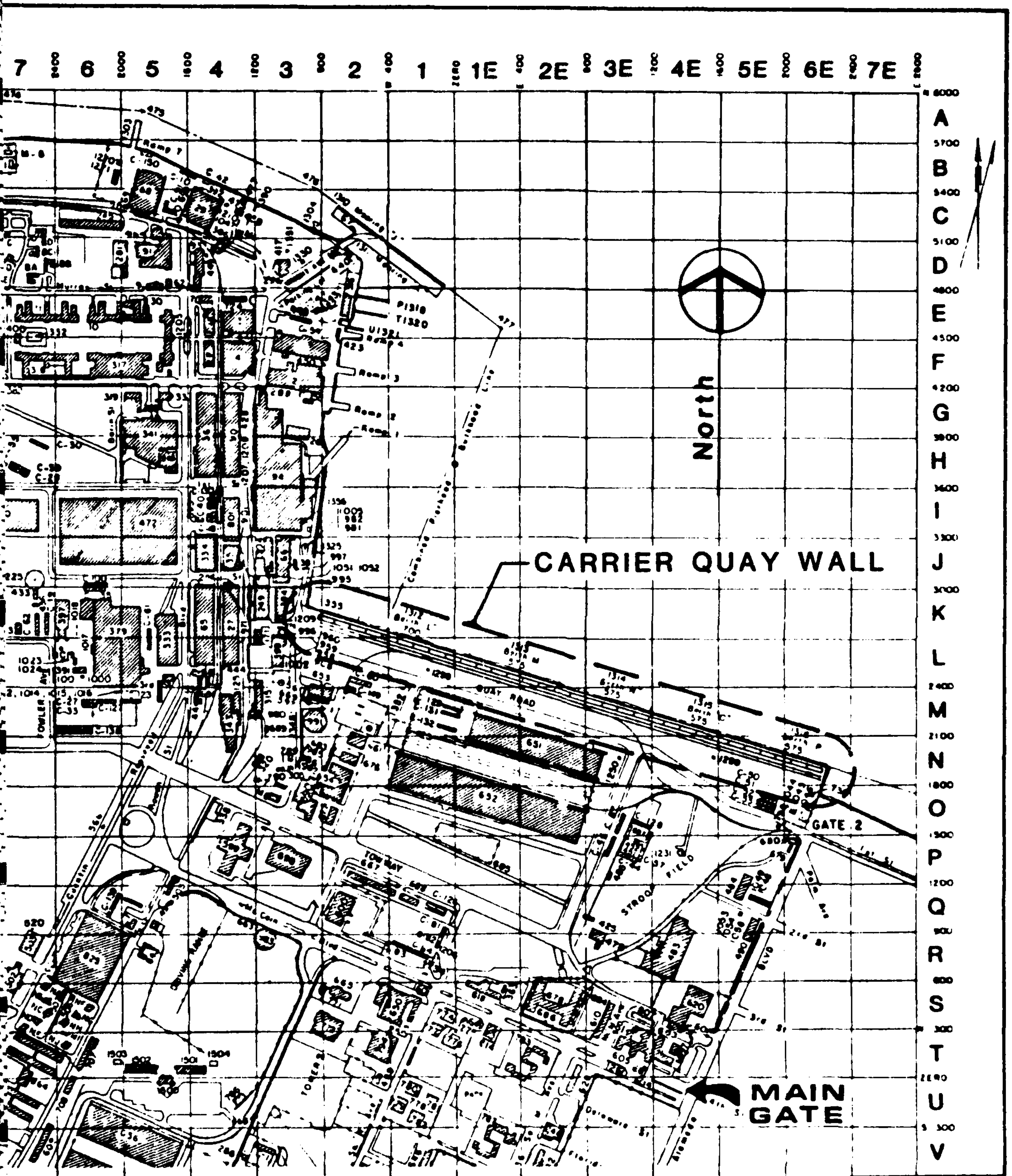
F RD WEST



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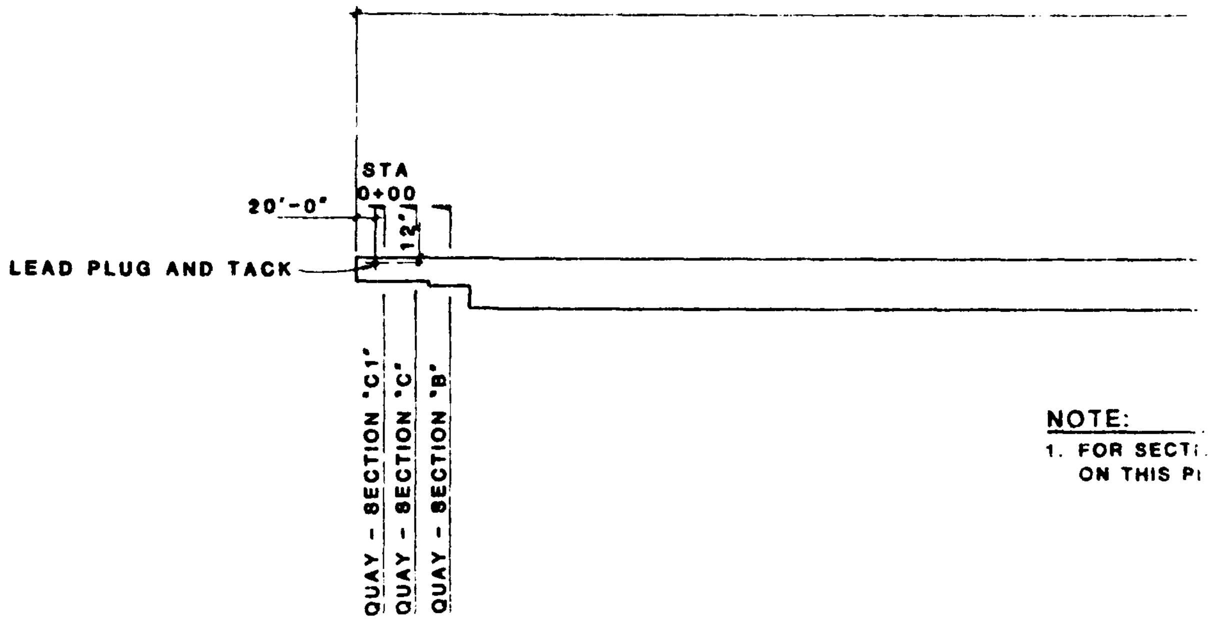
North
CARRIER QUAY WALL
QUAY ROAD
GATE 2
MAIN GATE

FIG. 5



NOTE:

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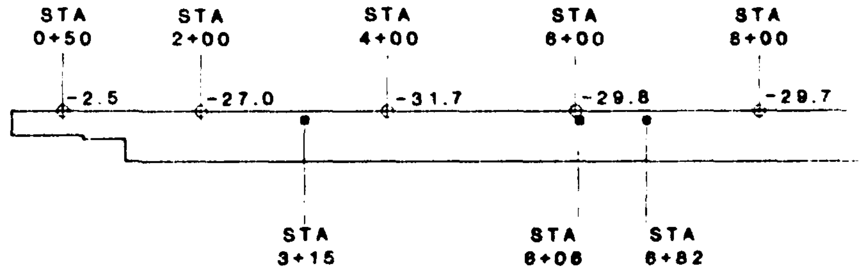

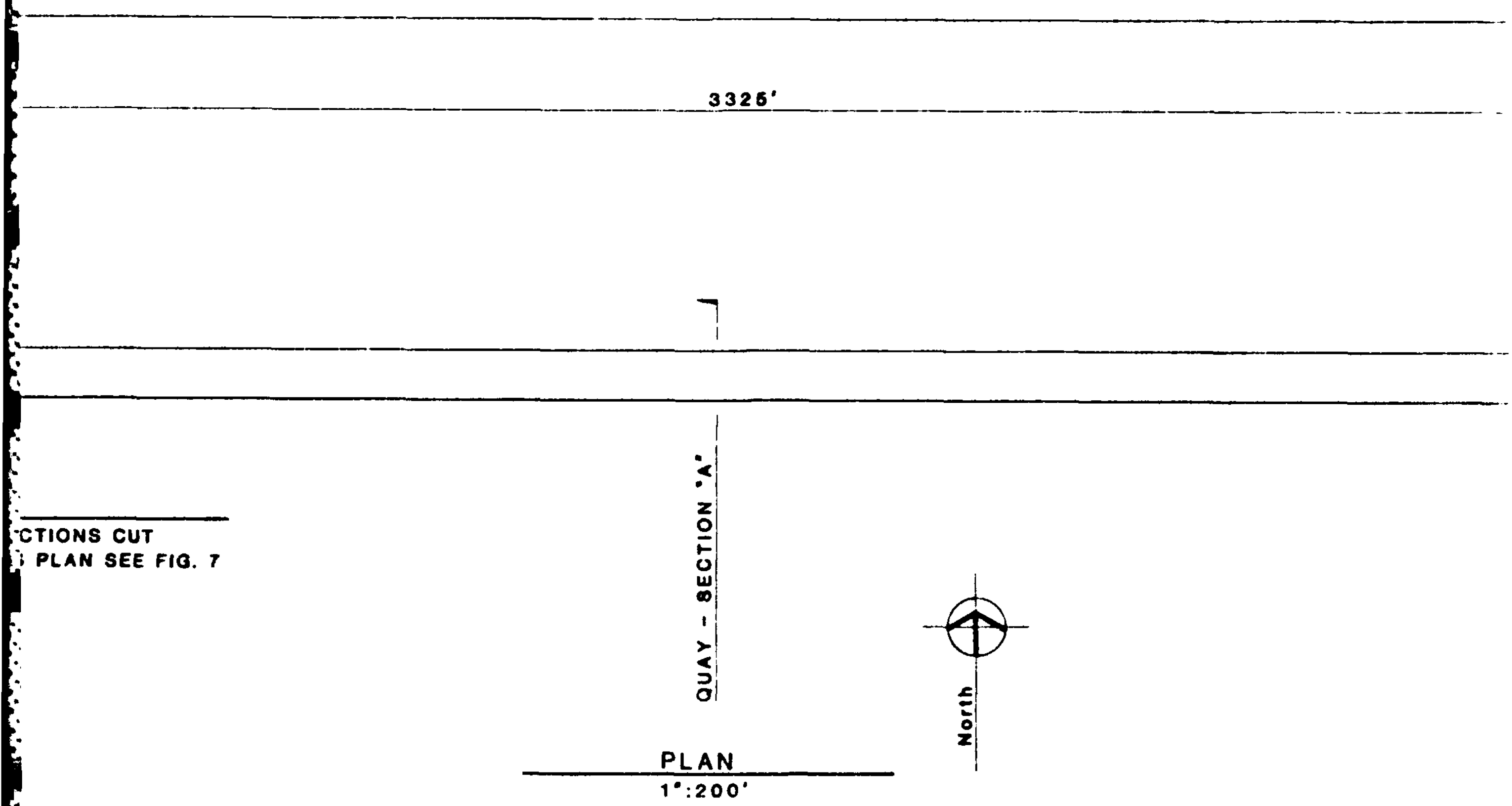
3325'
QUAY - SECTION 'A'
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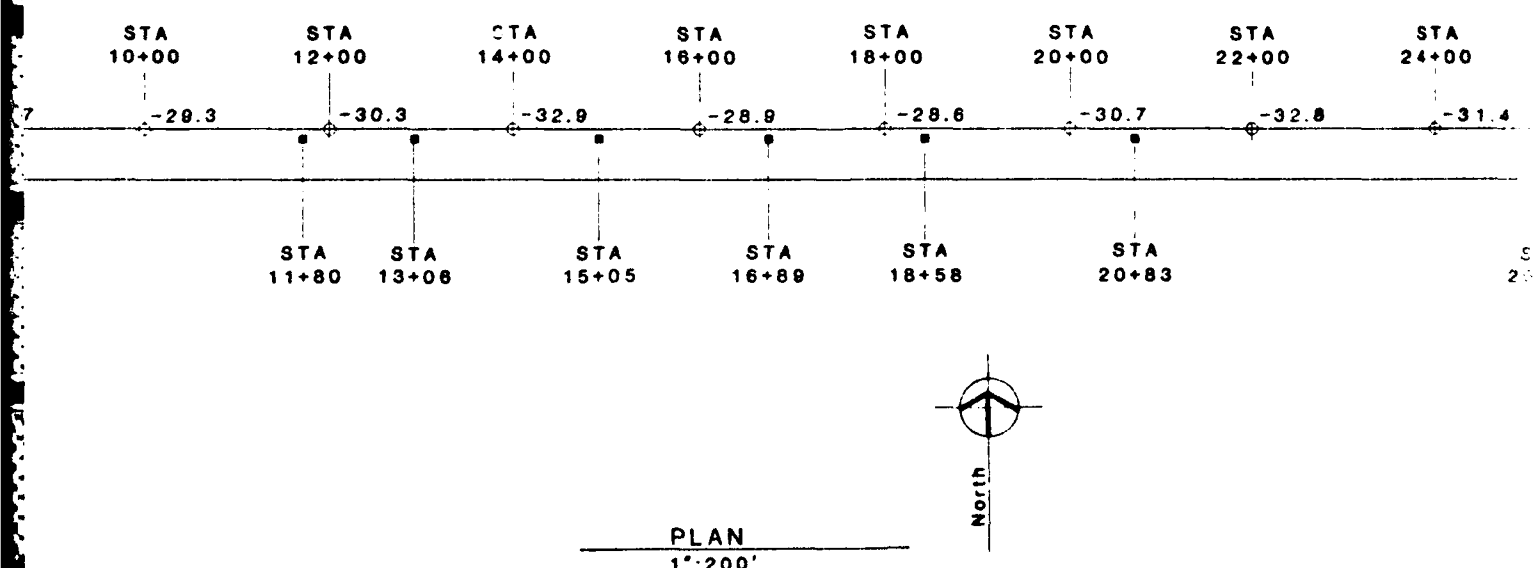
North
PLAN
1":200'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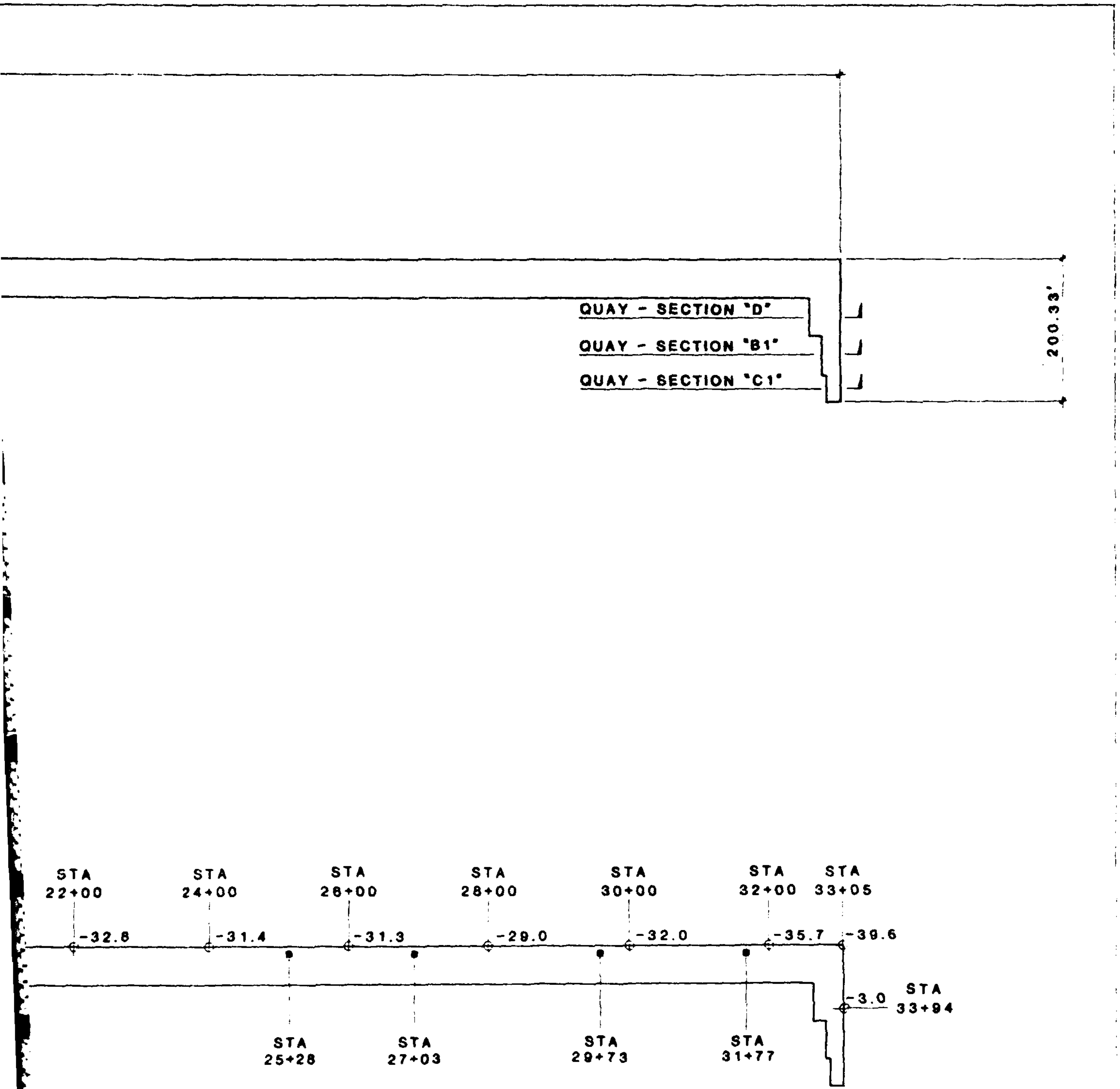


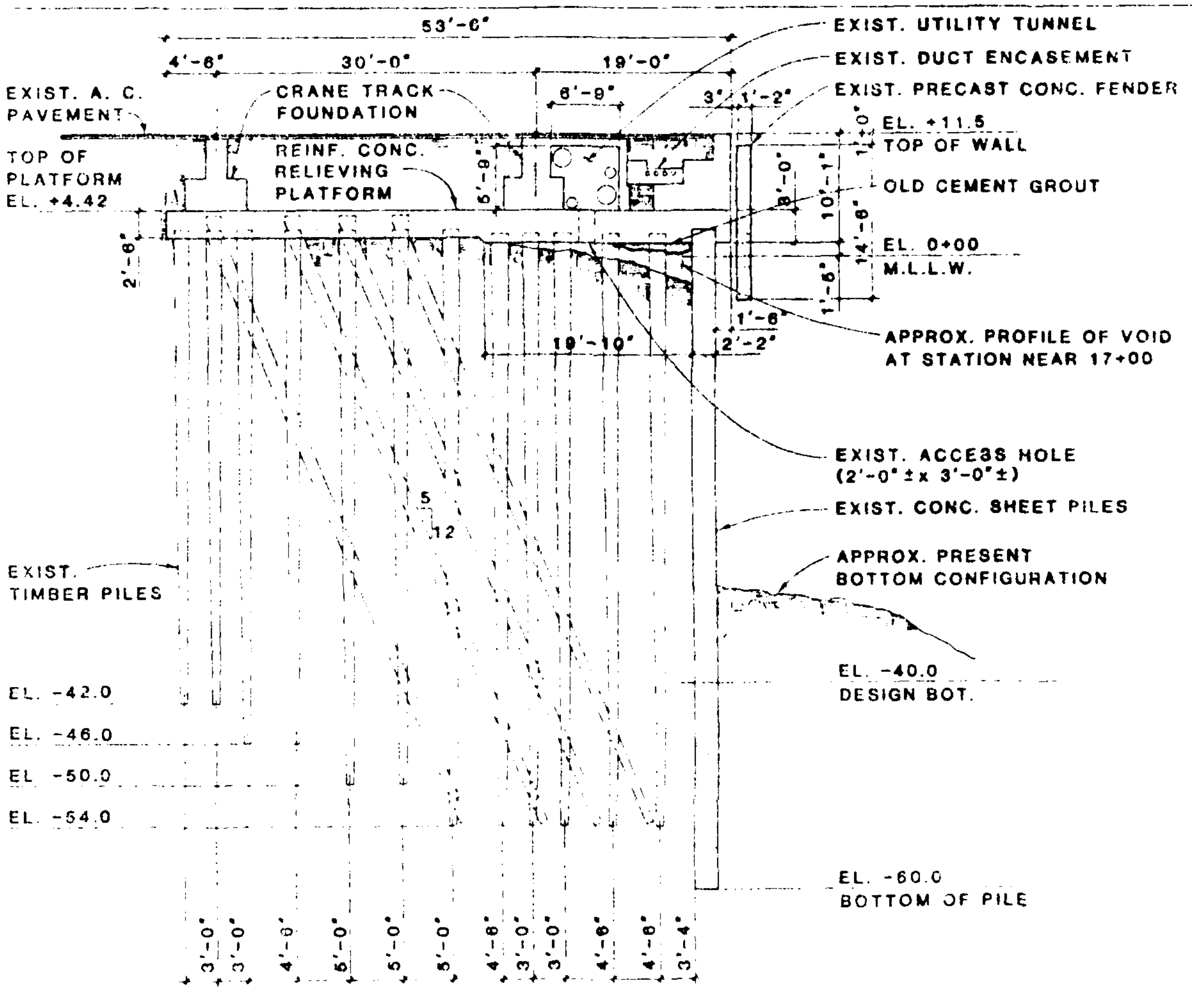
NOTES:

1. ● INDICATES EXISTING ACCESS HOLE THROUGH CONCRETE RELIEVING PLATFORM (2' x 3'±)

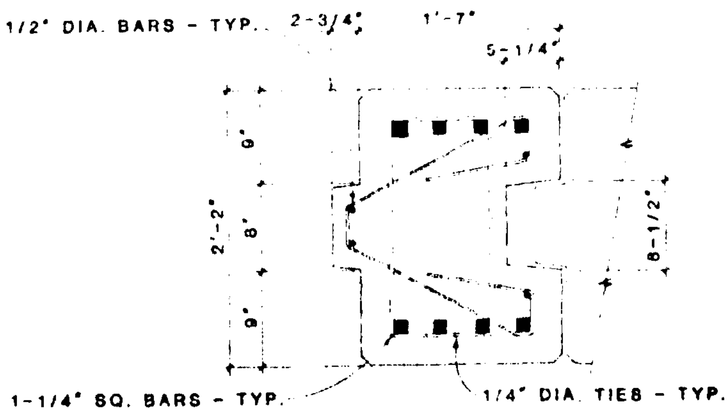
2. ○ INDICATES CLEANED LOCATIONS FOR LEVEL II INSPECTION

3. -2.6 INDICATES MUD LINE ELEVATION, MEAN LOWER LOW WATER DATUM - EL. 0+00

CARRIER QUAY WALL PLANS NAVAL AIR STATION - NORTH ISLAND, SAN DIEGO, CALIFORNIA		
Blaylock-Willis and Associates STRUCTURAL ENGINEERS SAN DIEGO, CALIFORNIA	DATE: OCT. 1984	FIG. 6



QUAY - SECTION "A"
1/16":1'-0"



TYP. PILE - TYPE "A"
3/4":1'-0"

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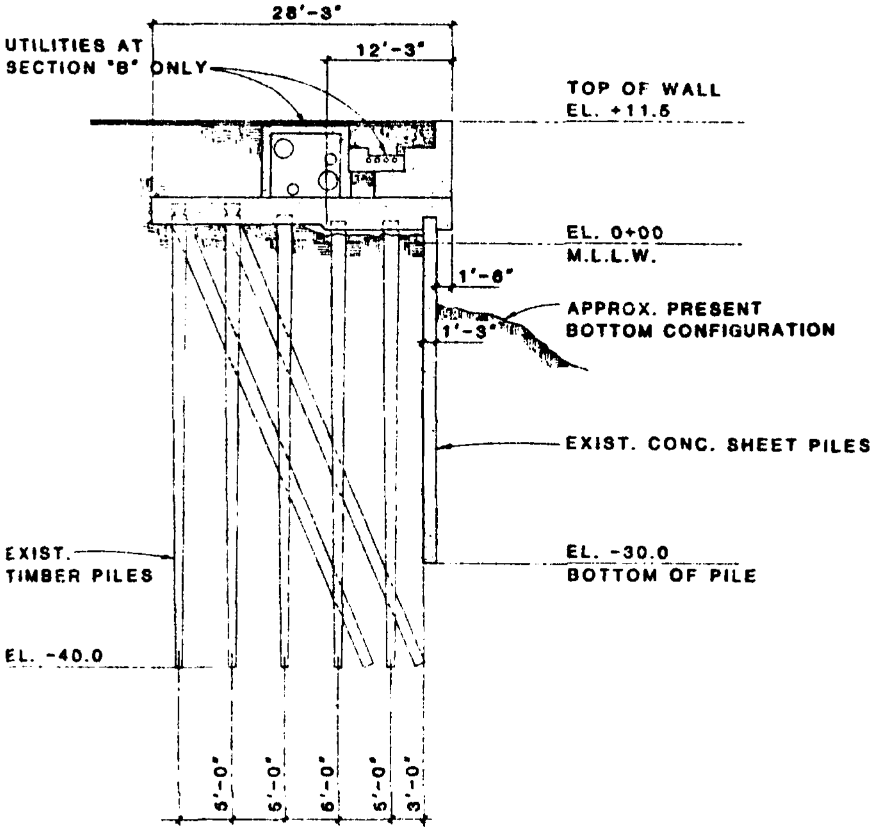
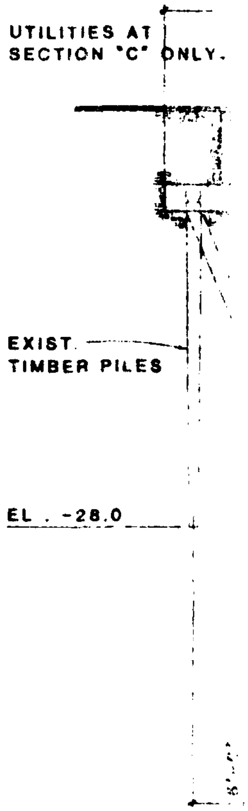
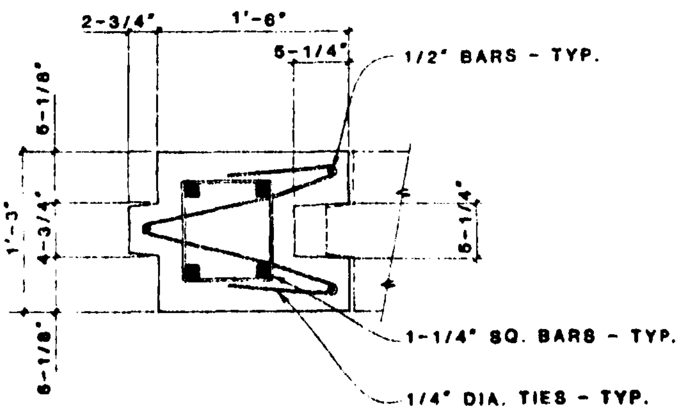


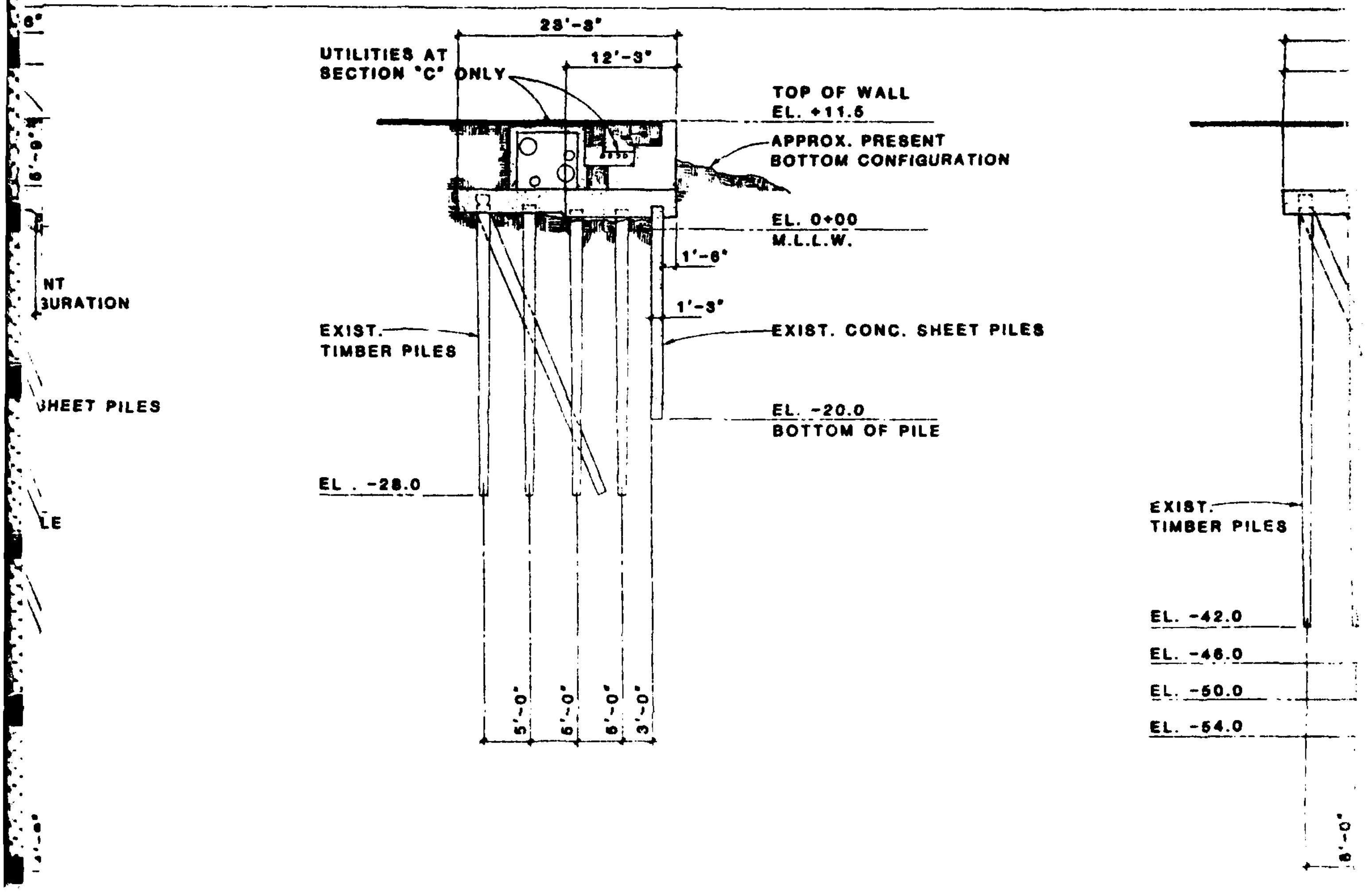
QUAY - SECTION "B" & "B1"
1/16":1'-0"



QUAY - SE
1/16":1'-0"

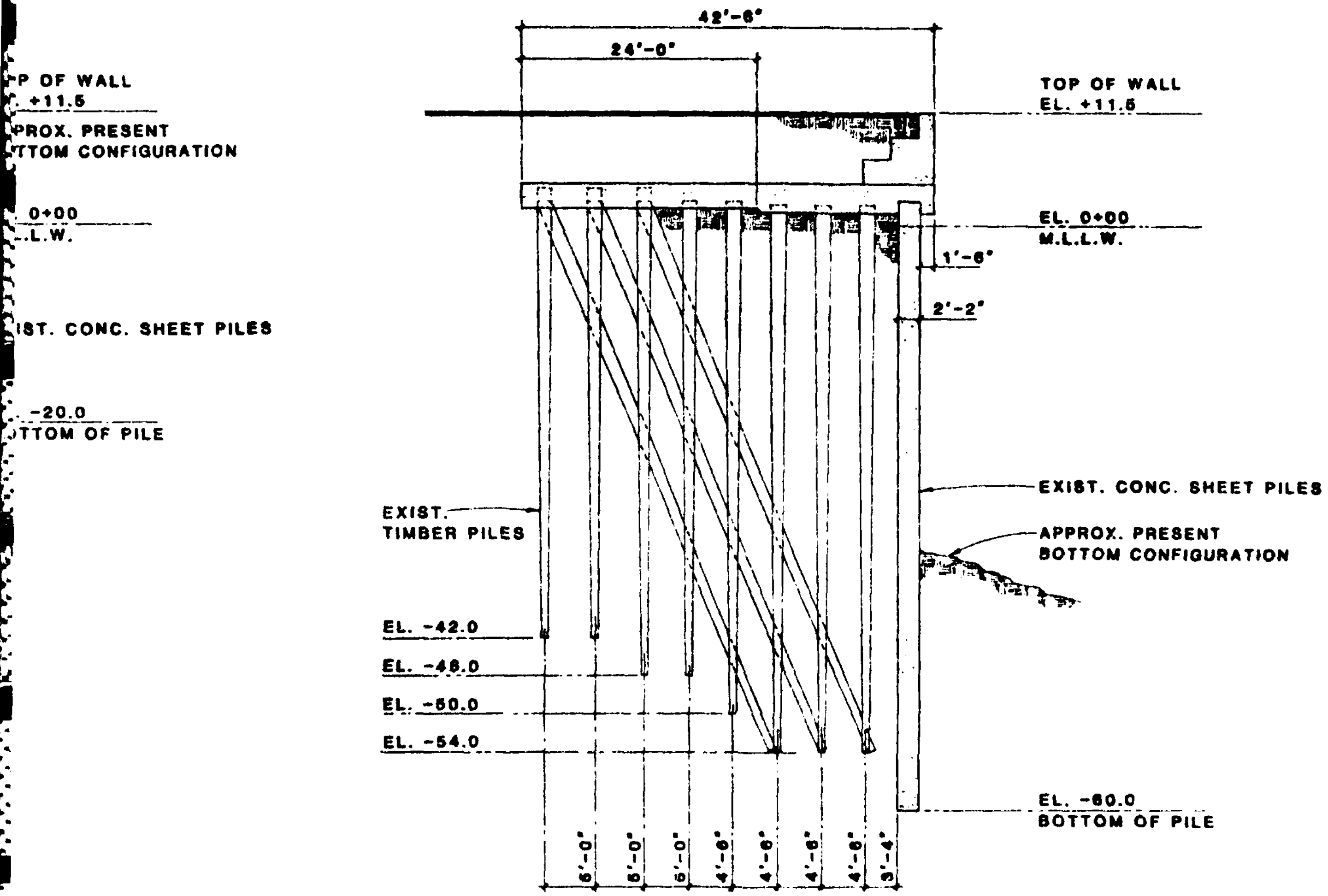


TYP PILE - TYPE "E" & "G"
3/4":1'-0"



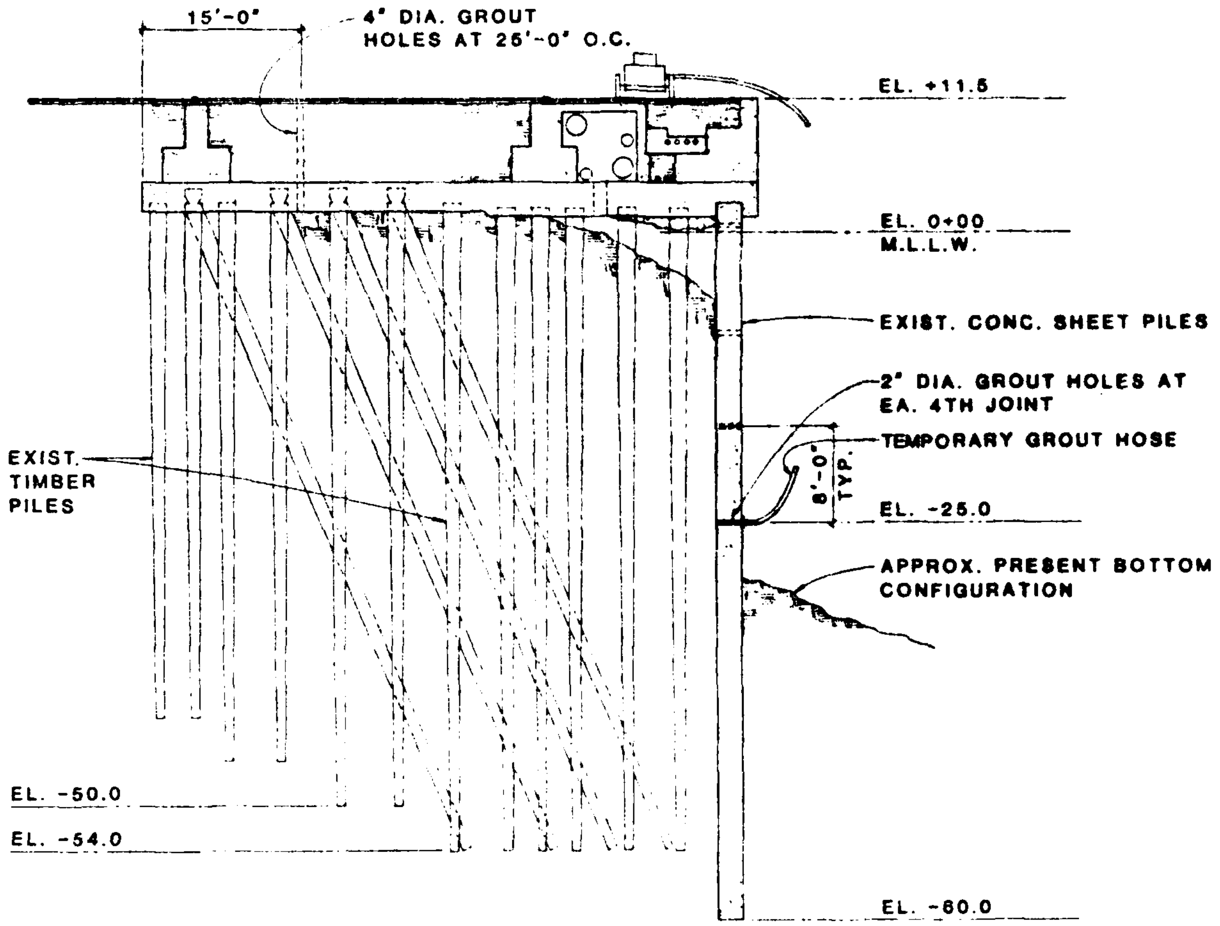
QUAY - SECTION "C" & "C1"

1/16":1'-0"



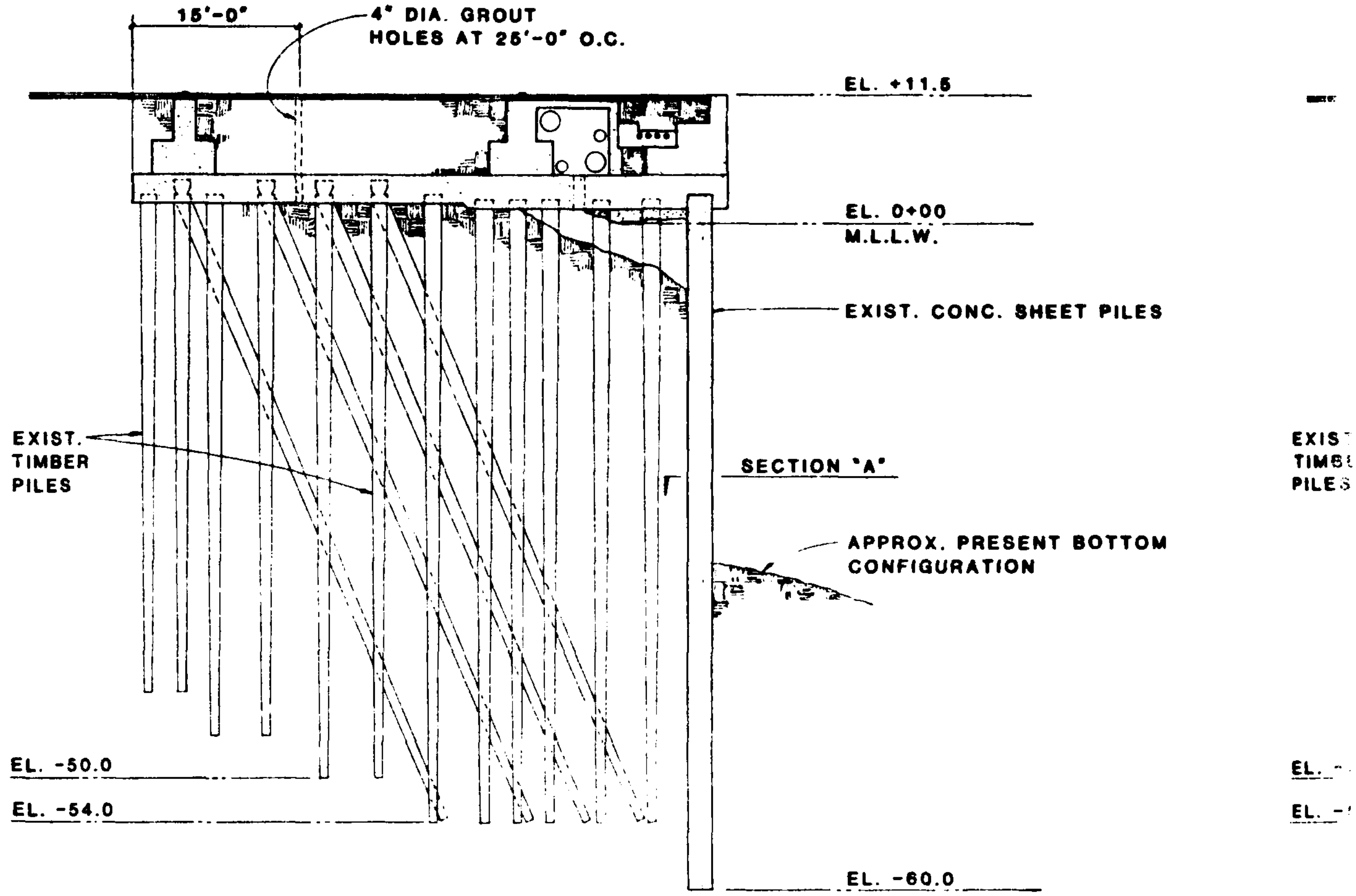
QUAY - SECTION "D"
1/16":1'-0"

CARRIER QUAY WALL TYPICAL SECTIONS NAVAL AIR STATION - NORTH ISLAND, SAN DIEGO, CALIFORNIA		
Blaylock-Willis and Associates STRUCTURAL ENGINEERS SAN DIEGO, CALIFORNIA	DATE: OCT. 1984	FIG. 7



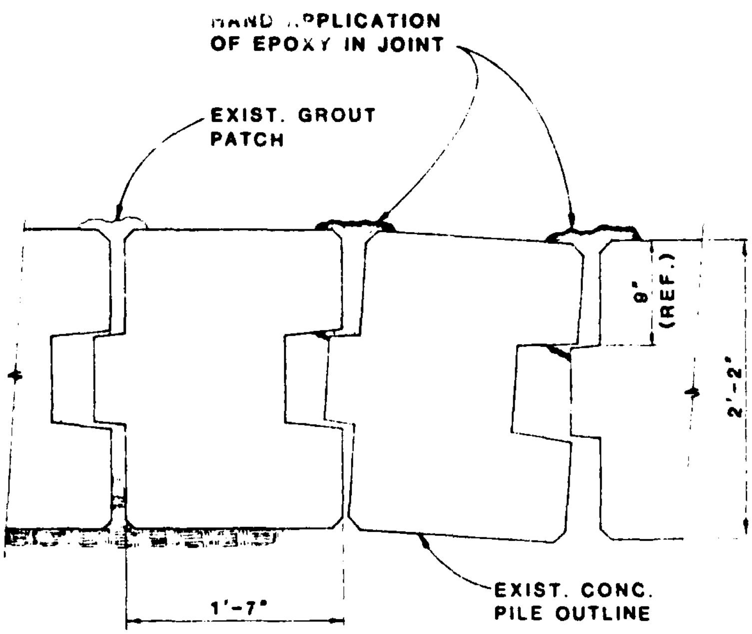
TYPICAL SECTION - METHOD No. 1

1/16": 1'-0"



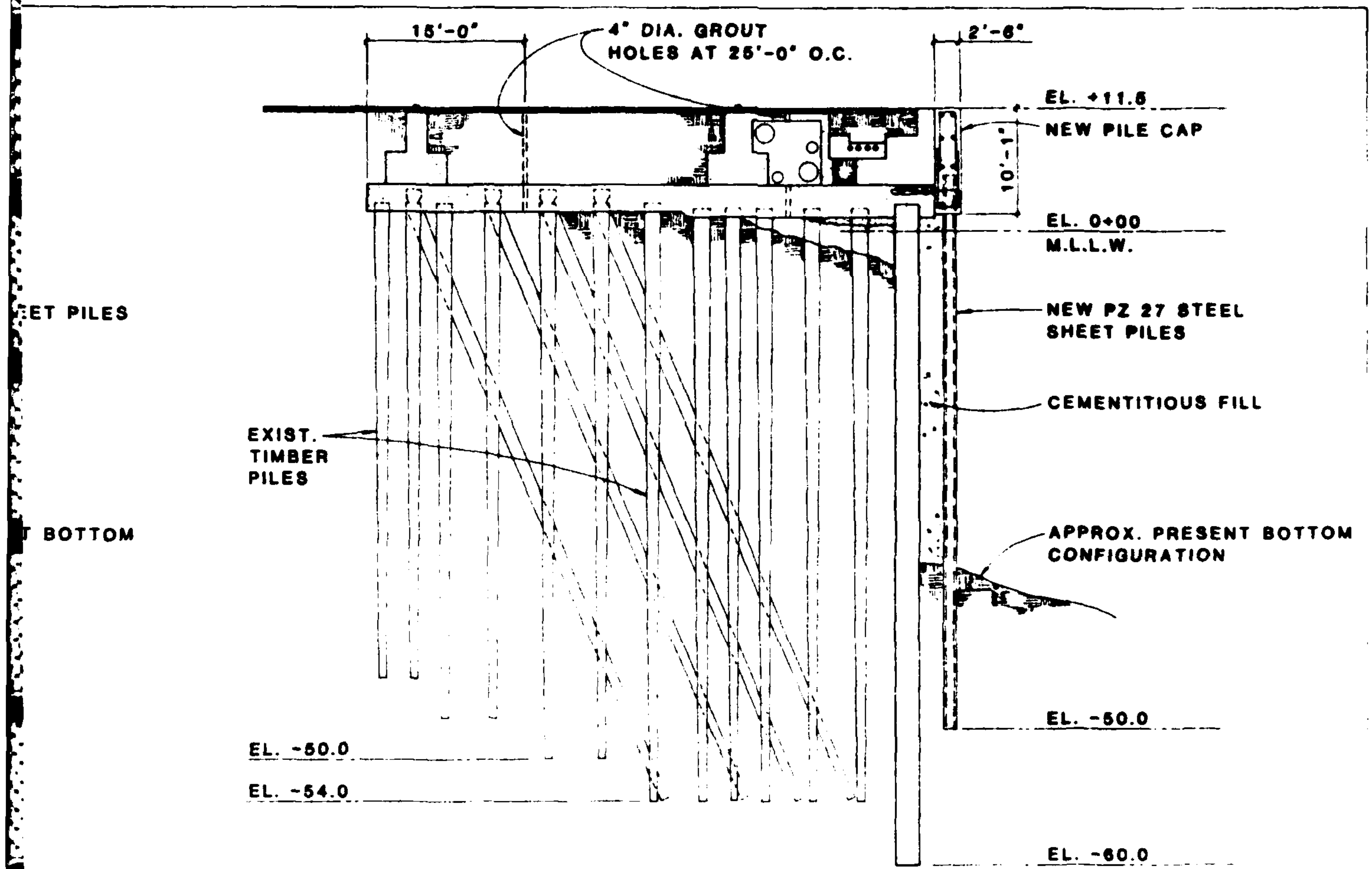
TYPICAL SECTION - METHOD No. 2

1/16": 1'-0"



PLAN - SECTION "A"

HAND SEAL APPLICATION 3/4": 1'-0"

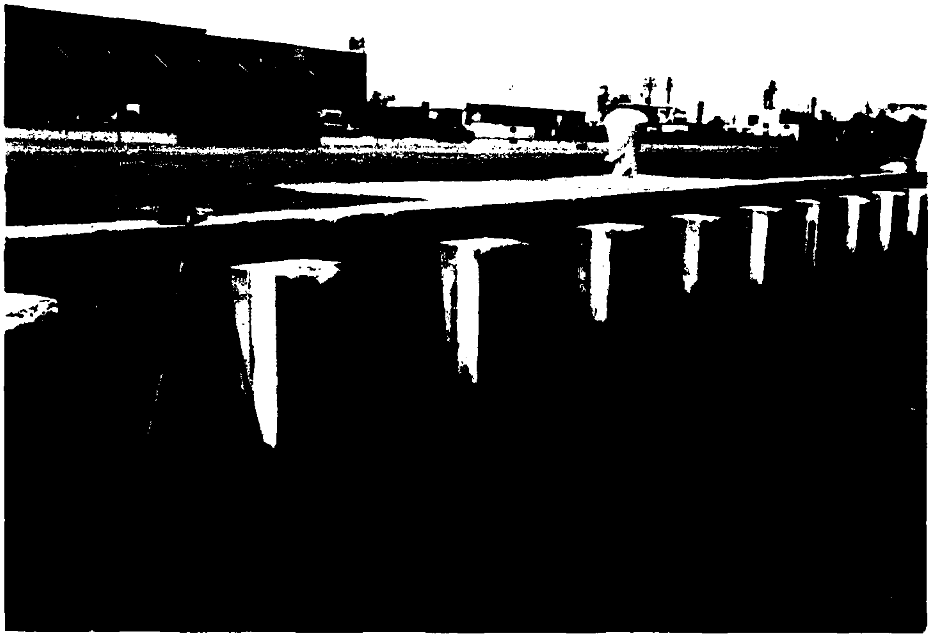


TYPICAL SECTION - METHOD No. 3

1/16": 1'-0"

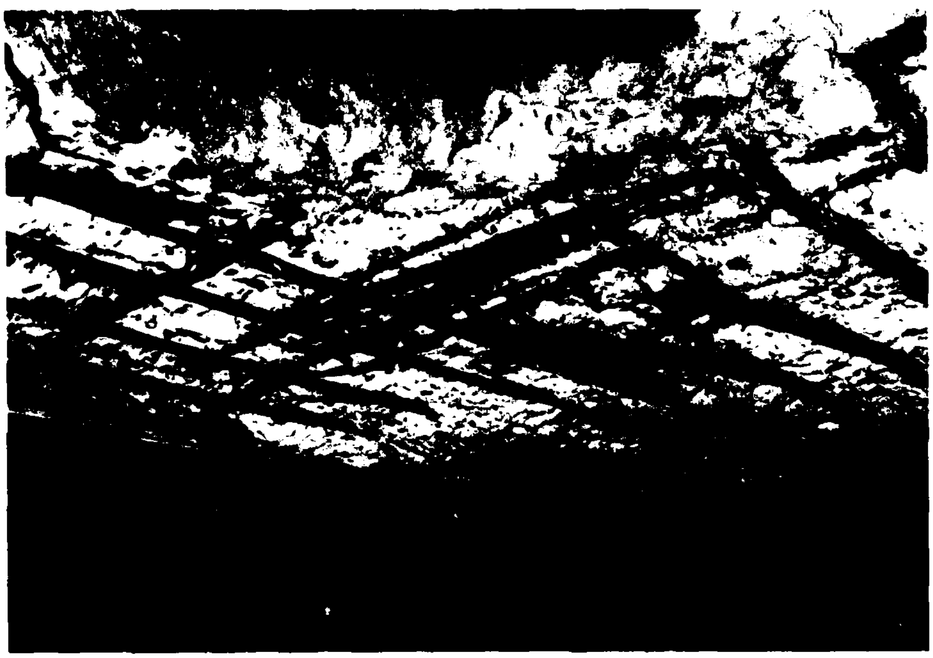
CARRIER QUAY WALL ALTERNATIVE REPAIR METHODS NAVAL AIR STATION - NORTH ISLAND, SAN DIEGO, CALIFORNIA		
Blaylock-Willis and Associates STRUCTURAL ENGINEERS SAN DIEGO, CALIFORNIA	DATE: OCT 1984	FIG. 8

9. Carrier Quay Wall, view is to the East from about Station 16+00.

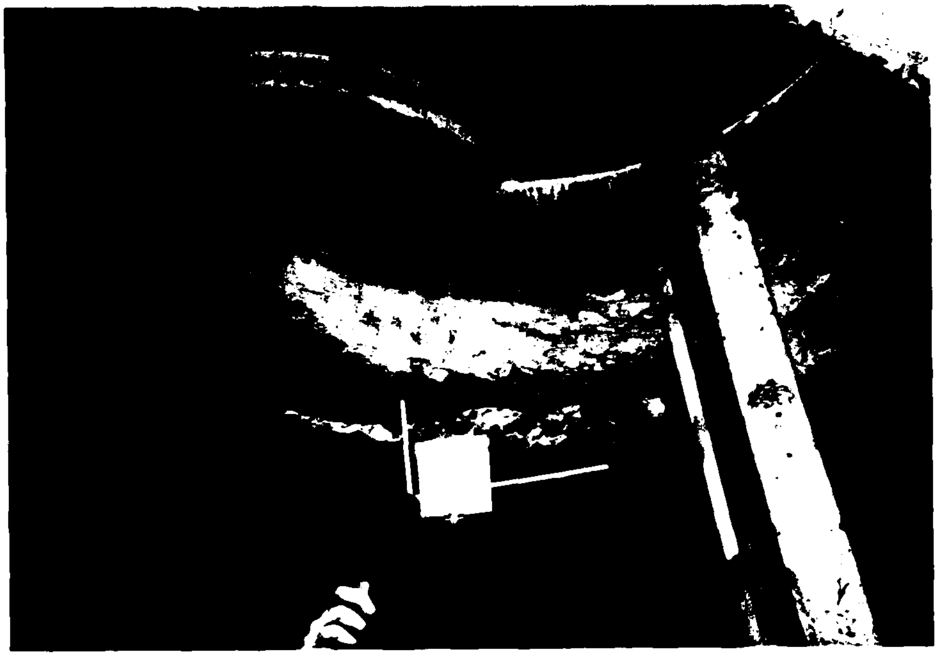


10. Carrier Quay Wall, view is to the East from about Station 16+00.

11. Carrier Quay Wall, in utility tunnel at Manhole #8 showing spall areas and rusted reinforcing steel. The capacity of the tunnel ceiling slab to support loads is seriously reduced.

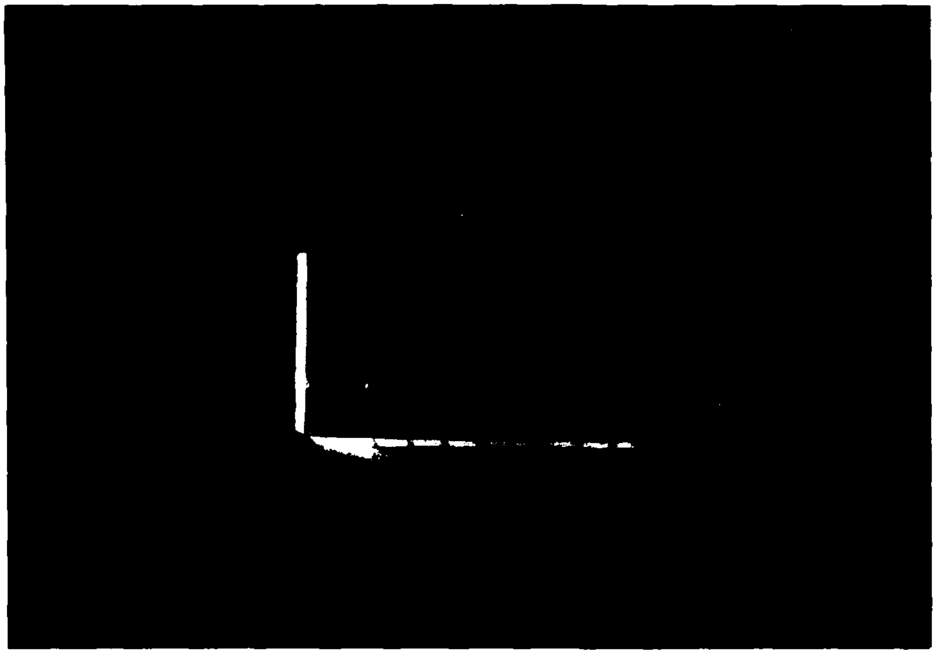


12. Carrier Quay Wall, in utility tunnel. View is to the southeast from Manhole #8. Plywood in background has been placed to prevent spalled concrete from falling on workers in the tunnel.

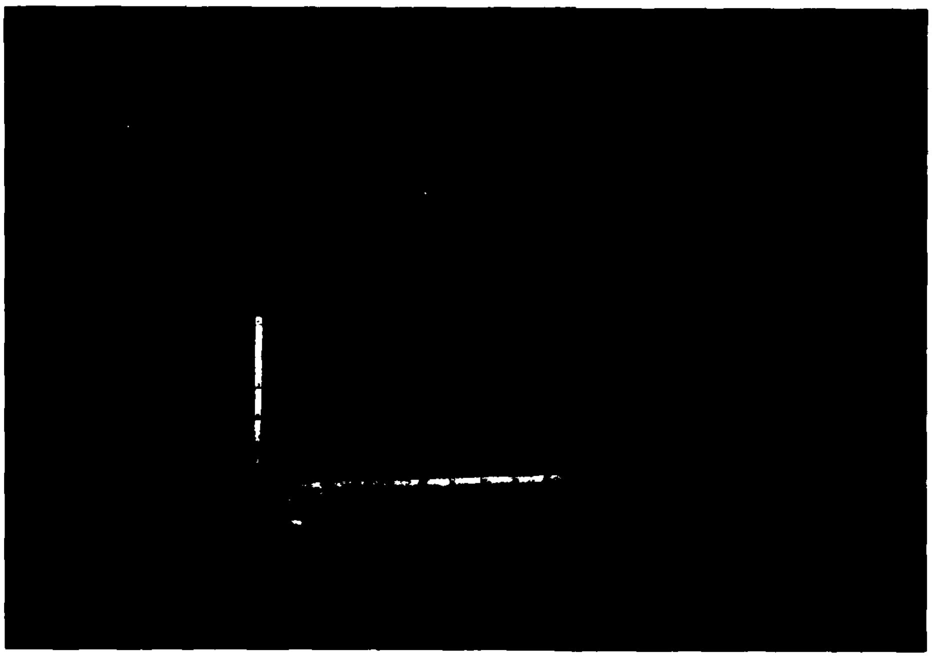


13. Carrier Quay Wall, in utility tunnel at Manhole #9. Rusting reinforcing steel has caused cracking and spalling in concrete ceiling slab.

14. Carrier Quay Wall, in utility tunnel. Ceiling spall in vicinity of Manhole #9.

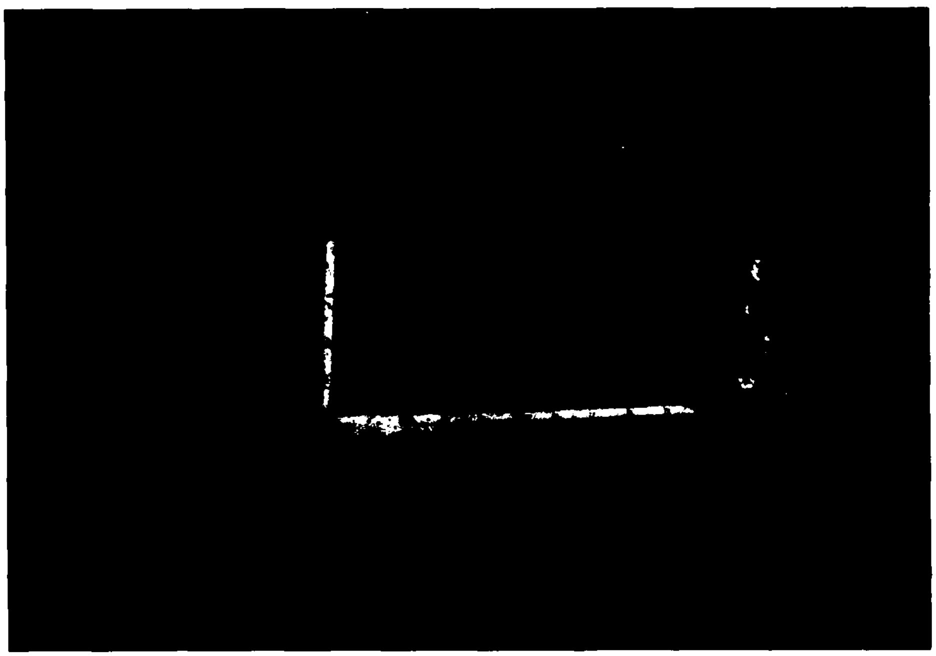


15. Carrier Quay Wall, at Station 16+00. Cleaned areas are shown at top of concrete sheet piles intersection with cast in place cap. The piles exhibit slight sulphate damage not considered serious.

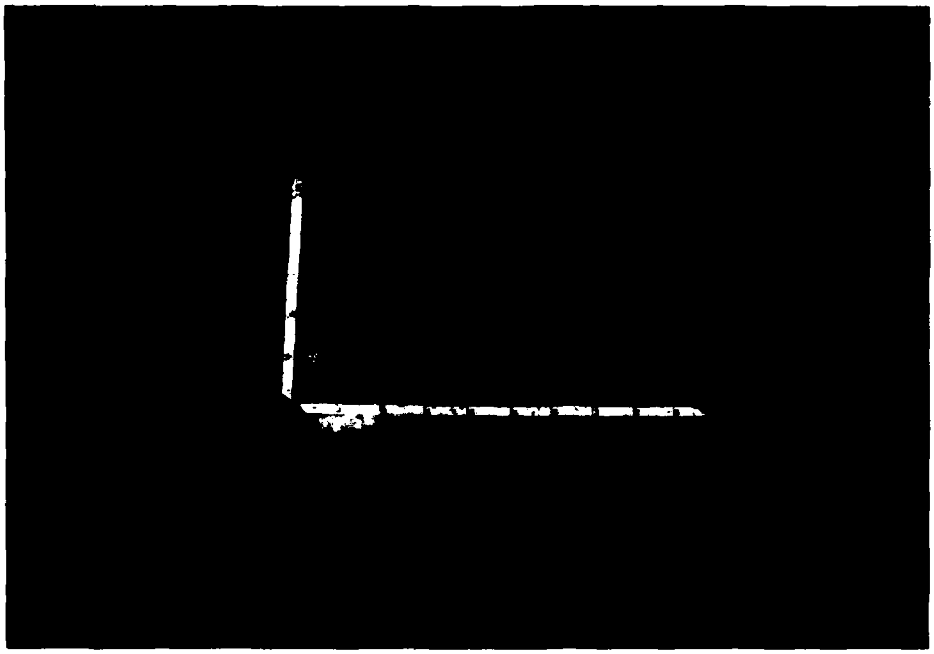


16. Carrier Quay Wall, at Station 16+00. Cleaned area is at the bottom adjacent to mudline.

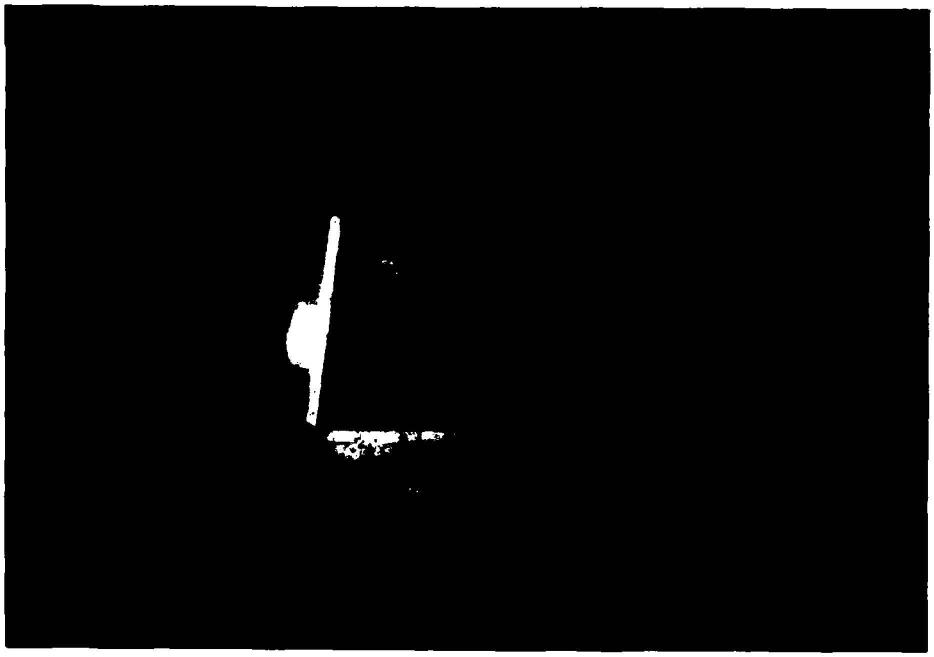
17. Carrier Quay Wall, at Station 16+45. The pyramid of soil is characteristic of leaking joint in concrete sheet pile wall.



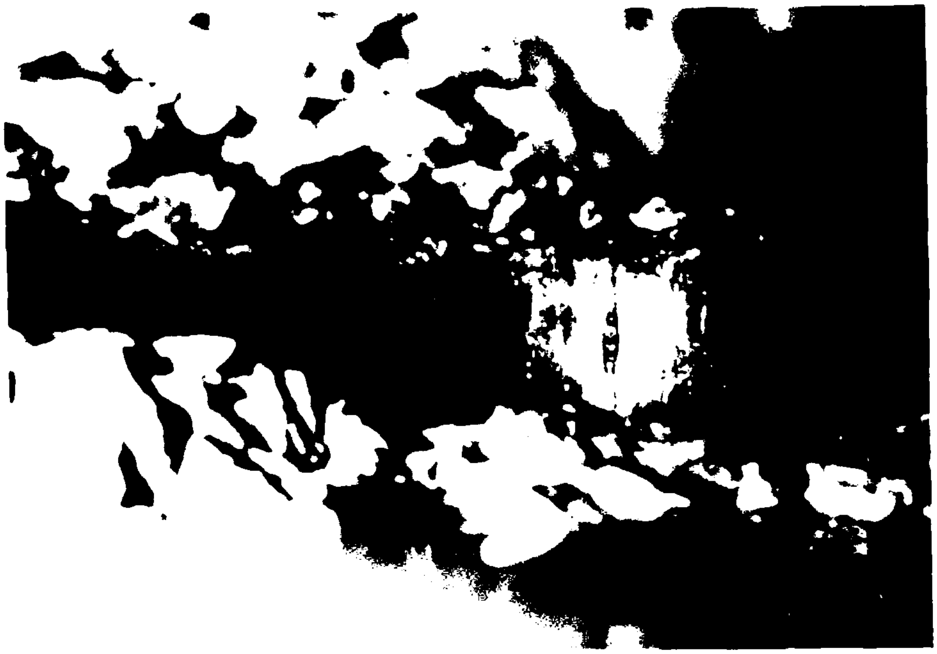
18. Carrier Quay Wall, Station 18+25. Joint shown is very wide due to one pile rotating during driving.



19. Carrier Quay Wall, Station 16+45. Picture shows old burlap packing used to seal joint leaking grout during 1970 grouting operation.



20. Carrier Quay Wall, at Station 16+45. Showing wide joint at mid-height of wall.



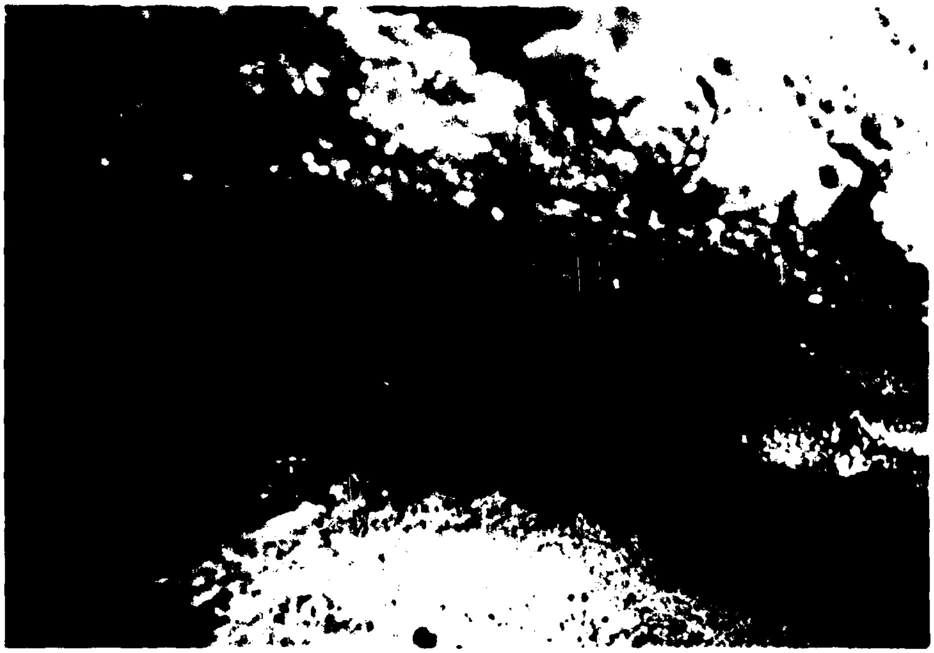
21. Carrier Quay Wall, approximately Station 31+75. Picture was taken at an access opening through the relieving platform in the utility tunnel. View is toward the water, showing exposed timber pile with very little borer damage.

22. Carrier Quay Wall, approximately Station 31+75. Picture taken through same access opening as Photo 21. View is toward Northwest. The concrete sheet piling are about 12' away. The void is similar to a beach sloping toward the water.

23. Carrier Quay Wall, approximately Station 17+00. Showing access hole through the relieving platform in the utility tunnel. Hole was made in 1967.

24. Carrier Quay Wall, approximately Station 17+00. Showing straight down view of access hole. The void or space between the bottom of the relieving platform and the soil below is about 12 inches at the hole. This dimension increases toward the water.

25. Carrier Quay Wall, approximately Station 17+00. Picture shows the underside of the relieving platform at the top of a wooden pile. The surface of the pile shows damage from marine borers.

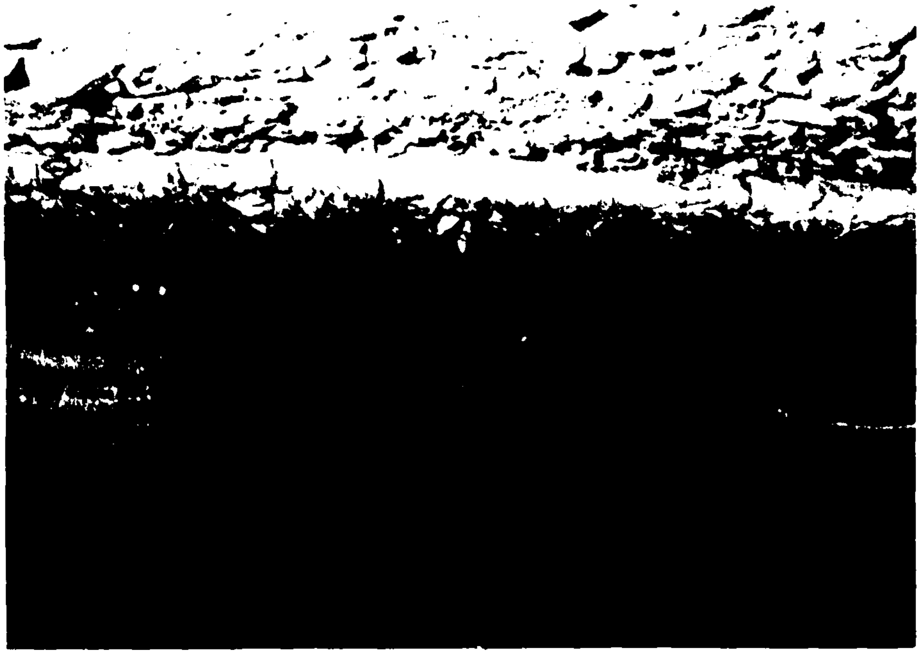


26. Carrier Quay Wall, approximately Station 17+00. View is toward the Northwest. There is about 12" vertical void dimension at the access hole, insufficient to gain access to the deeper void toward the water.

27. Carrier Quay Wall, approximately Station 17+00. View is away from the water where the beach slopes upward and intersects the bottom of the relieving platform a few feet from the access hole. Pile in the foreground has suffered marine borer damage.

28. Carrier Quay Wall, 1967 photo, taken in void below the relieving platform of front row pile near Station 17+00. Note deep ring of damage at top of pile. Center of pile shows exposed toredo burrows exposed by limnoria damage. Void filled in 1970 repair.

29. Carrier Quay Wall, 1967 photo, taken in void below relieving platform of typical front row pile. Divers hand gives scale to depth of damage. Void filled in 1970 repair.



30. Carrier Quay Wall, 1967 photo, taken of void below relieving platform. View looking south (inboard) from inside face of concrete sheet piling. The "beach" emerges from the water in the background. Void filled in 1970 repair.

31. Carrier Quay Wall, 1967 photo, taken below relieving platform. View looking west along inside face of concrete sheet piles. Berm in background has deeper void behind it. Void filled in 1970 repair.

4.3 PIER BRAVO

4.3.1 DESCRIPTION OF THE FACILITY

Pier Bravo at the Naval Air Station is located at the west end of the island adjacent to the entrance to San Diego Bay. It was contructed in two increments. The first increment consists of the access Pier, 75 feet wide by 191 feet long and the working Pier, 75 feet wide by 625 feet long. This increment was built in 1975-1976 under construction contract N62474-74-C-3755. The working Pier is symmetrical about the access Pier forms a "T" shape, and is oriented North and South.

The second increment consists of 327.5 foot extensions both North and South of the working Pier. These extensions are not solid piers as is the working Pier but are each two 30 foot by 75 foot platforms connected by 12 foot by 133'-9" ramps. The second increment was constructed under construction contract N62474-77-C-2166 in 1979.

Pier Bravo has not been the subject of an underwater inspection previous to this time.

The earlier increment of the Pier is supported on 24 inch octagonal prestressed concrete piles. The second increment is supported by both 20 inch square piles and 18 inch octagonal piles, of prestressed concrete.

4.3.2 OBSERVED CONDITIONS

The pier is in excellent condition. The piles, the pile caps and deck surfaces show no evidence of deterioration. Even expected surface shrinkage cracking in the concrete deck is considered less than normally expected.

The marine growth on the structural piles is very healthy and difficult to remove. No single fault could be found with any concrete pile.

4.3.3 STRUCTURAL CONDITION ASSESSMENT

Increment one was designed to the following structural criteria:

A. Ammunition Pier

Class of Ship:

1. Destroyer (OLG)

Full Load Displacement	= $9{,}000^{LT}$ ($20{,}160^{K}$)
Velocity of Approach	= 1.0 Knot (1.15 MPH)
Angle of Approach	= 20^{o}

2. Carrier (CVA) - For future reference only

Full Load Displacement	= $85{,}350^{LT}$ ($191{,}184^{K}$)
Velocity of Approach	= 0.3 Knot (0.345 MPH)
Angle of Approach	= 10^{o}
Current Velocity	= 0.7 Knot (0.81 MPH)
Wind Velocity	= 50 MPH
Waves	= None
Top of Concrete Deck	= + 14 Ft.
Bottom of Dredged Elev.	= -37 Ft. (-42 Ft. future)
Datum	= MLLW = 0.0 Ft.

B. DECK LOADING

	LOAD OVER REAR	LOAD OVER SIDE
1. $2\text{-}25^{T}$ Tr. Cranes:	12^{K} on tires	None on tires
	12^{K} +DL on O-Rig.	12^{K} +DL on O-Rig.

2.	$1 - 40^T$ Tr. Crane:	48.6^K on tires 80^K on O-Rig. w/DL on tires	39.5^K on tires 80^K on O-Rig. w/DL on tires

3. Uniform Live Load = 600 PSF

C. 24" OCTAGONAL SOLID PRESTRESSED CONCRETE PILES

Downward Load Capacity	= 267 Tons (@ refusal)
Uplift Capacity	= 20T @ tip elev. -70'
	= 40T @ tip elev. -80'
Unsupported Length (Max.)	= 69 Ft. (Fixed top & bot.)
Point of Fixity	= 18 Ft. below mudline

D. CONCRETE DECK

Prestressed Concrete	f'c	=	5,000 PSI
1/2" - 7-Wire Strands	f's	=	270,000 PSI
Concrete Topping	f'c	=	5,000 PSI
Reinforcing Bars	f's	=	24,000 PSI

E. DECK FITTINGS

Large Bollard	T	=	70,000 LBS.
42 in. Cleat	T	=	35,000 LBS.

F. SEISMIC - ZONE 4 (NAVFAC P-355)

$v = ZKCW$

Where: $Z = 1.5$ (High-Loss-Potential)
$K = 0.8$ (Dual Bracing System)
$C = 0.1$ (One Story Structure)
W = Total Dead Load Plus Transient Live Load (50% LL)

G. PILE DRIVING

Hammer type - single or double acting rated energy - 40,000 to 50,000 ft.-lbs.

Deviation from the above will require authorization from the O.I.C. and a new wave equation analysis to ensure that the piles will not be overstressed while being driven. Driving with full energy shall not be commenced until the pile has been jetted to depth and seated in dense sand.

H. PILE LENGTH

Pile length given is fabricated pile length.

GENERAL NOTES FOR PIER:

1. Piles shall have a minimum embedment of 30 feet below the dredged bottom, with a 1'-0" allowance for over dredging. Pile lengths on lines N & R are based on the future dredged elev. of -42 feet. Piles shall be jetted through very dense sand to within 5 feet of the required tip elevation, and shall then be driven to "refusal" without further jetting. "Refusal" (based on wave equation results) shall be defined as a minimum of 150 blows per foot from a single acting hammer with a rated energy range of 40,000 to 50,000 ft.-lbs.

2. All elevations shown on the drawings refer to mean lower low water datum; MLLW = EL.0.0'.

3. Rubber fenders, shall be capable of resisting a 30,000 lb. impact force with a 15,000 ft.-lb. energy absorption and a 12 inch deflection.

The second increment was designed to the following structural criteria:

CONCRETE

1. Concrete shall have strength and density as follows:

	f'c	Density (Air Dry)
A. Prestressed Concrete Piles:	6,000 PSI	150 PCF
B. All C.I.P. Concrete (U.O.N):	4,000 PSI	150 PCF
C. Precast, Prestressed Deck Members (f'ci MIN = 3,500 PSI):	5,000 PSI	115 PCF
D. Grout & Precast Cap Soffits:	4,000 PSI	150 PCF

DESIGN CRITERIA

1. 150 PSF uniformly distributed live load, or:

2. Unlimited operation of 11,000# fork lift with maximum wheel load: 3,000#. (Approximately equivalent to 3 ton rated fork lift w/o load.)

3. Wind Load: Based on wind velocity of 50 mph.

4. Seismic Load: V=ZKCW, Z=1.5, K=0.8, C=0.1, W= D.L.+25%L.L

PILE DRIVING

1. Piles shall be driven to the minimum tip elevations shown in table A/Sl, S2. In addition, piles shall be driven to an ultimate driving resistance of 660,000 lbs. as determined by the formula:

$$U \text{ (Ultimate driving resistance)} = \frac{aWH}{S+1/2P}$$

WHERE:

$$a = \text{Efficiency of Hammer} = \frac{\text{Energy Delivered}}{\text{Rated Energy}}$$

WH = Rated hammer energy in foot pounds

S = Penetration in feet per blow (average of last 10 blows)

$$P = \frac{2a\ WHL}{AE}$$

WHERE:

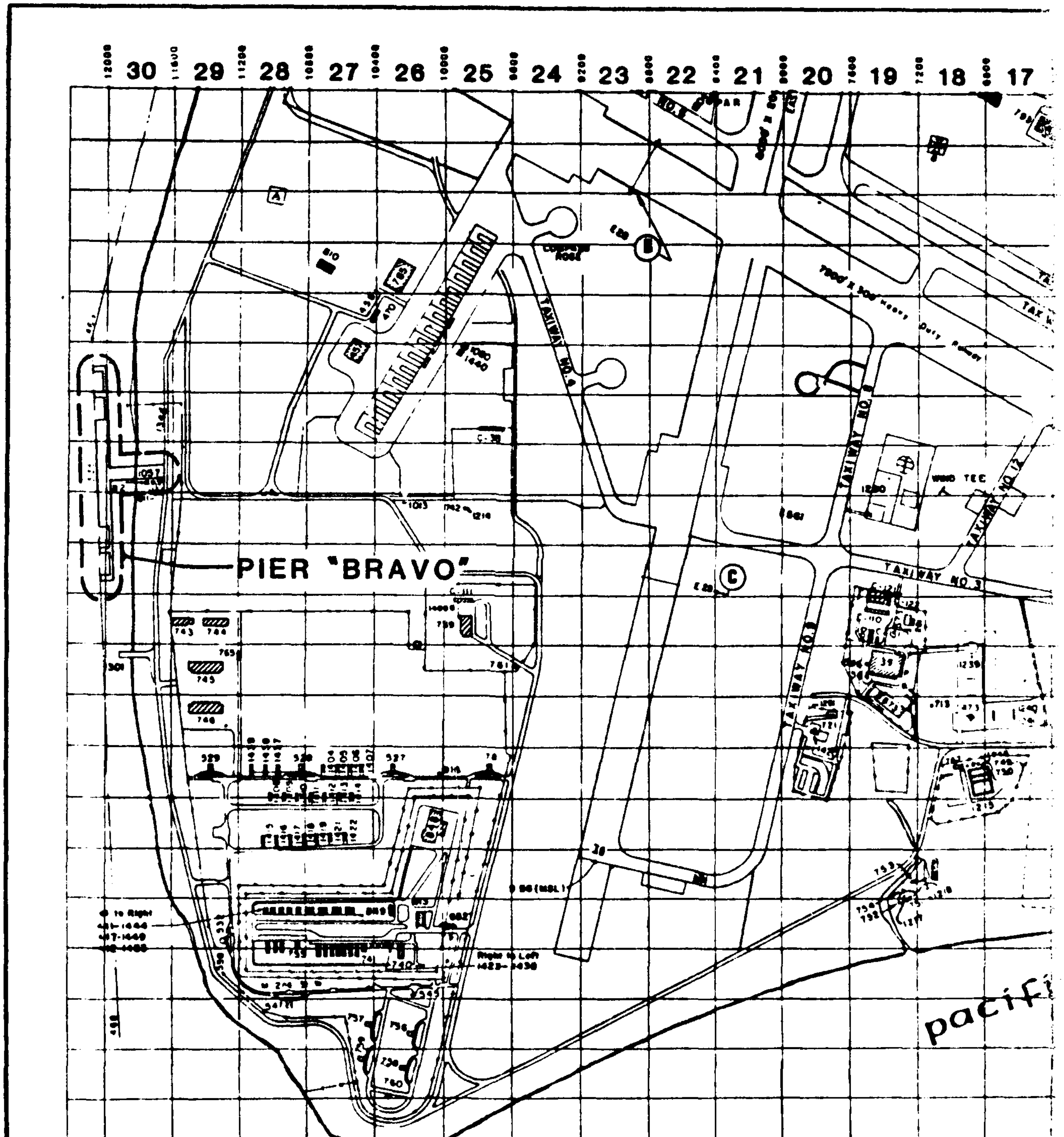
L = Length of pile in feet
A = Cross sectional area of pile in square feet.
E = Modulus of elasticity in pounds per square foot.

2. In the event that hard driving of 2 times the calculated blow count in blows per foot is reached within 3 feet of minimum tip, then driving may be discontinued after a minimum of 128 blows at the hard driving rate.

Simple calculations developed from the above design criteria show the Pier to be structurally adequate for any combination of the loads to which it can be subjected.

4.3.4 RECOMMENDATIONS

Pier Bravo is in excellent condition. The only recommendation is that the piles be inspected again in six years.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TAXIWAY NO. 5
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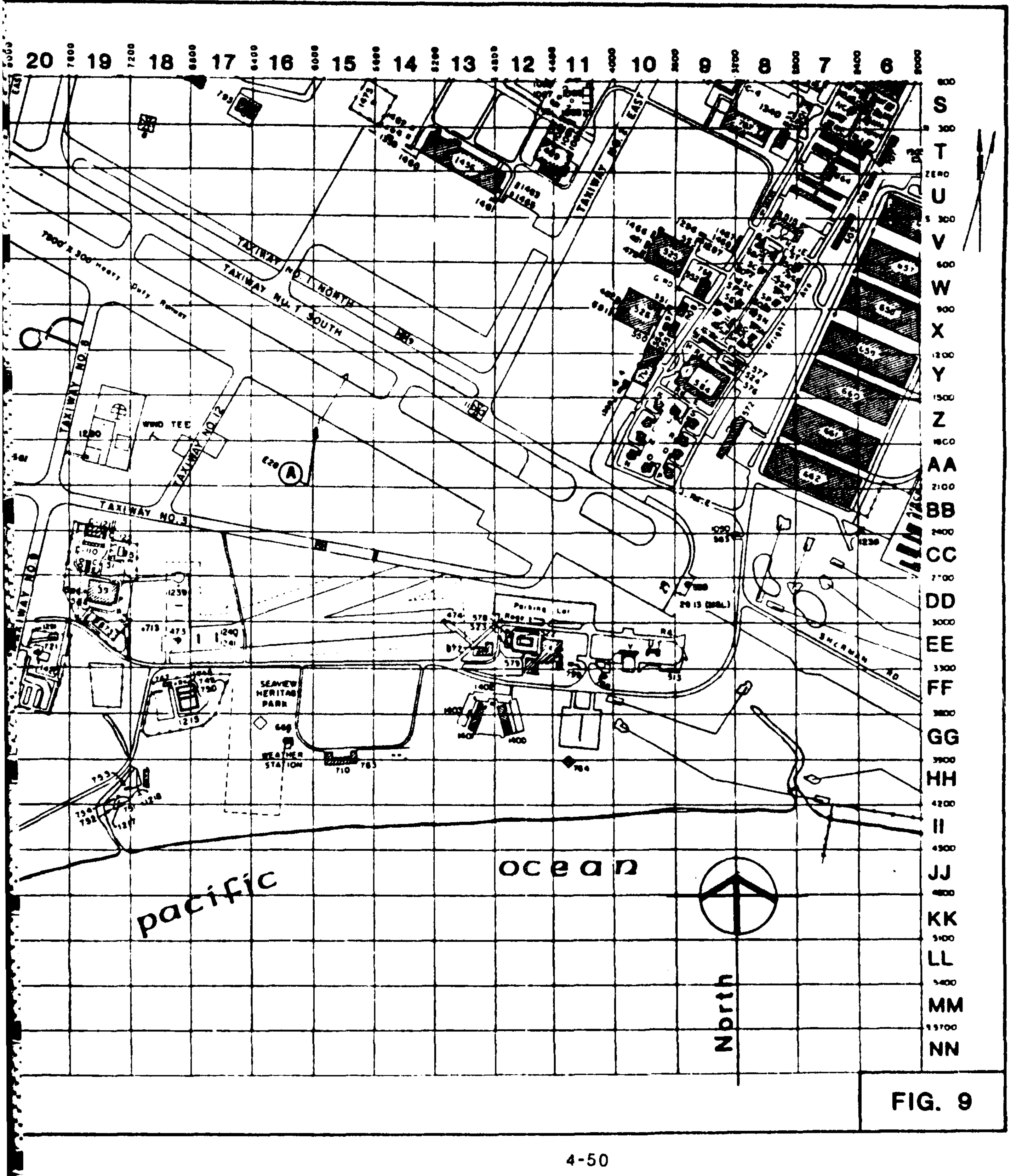

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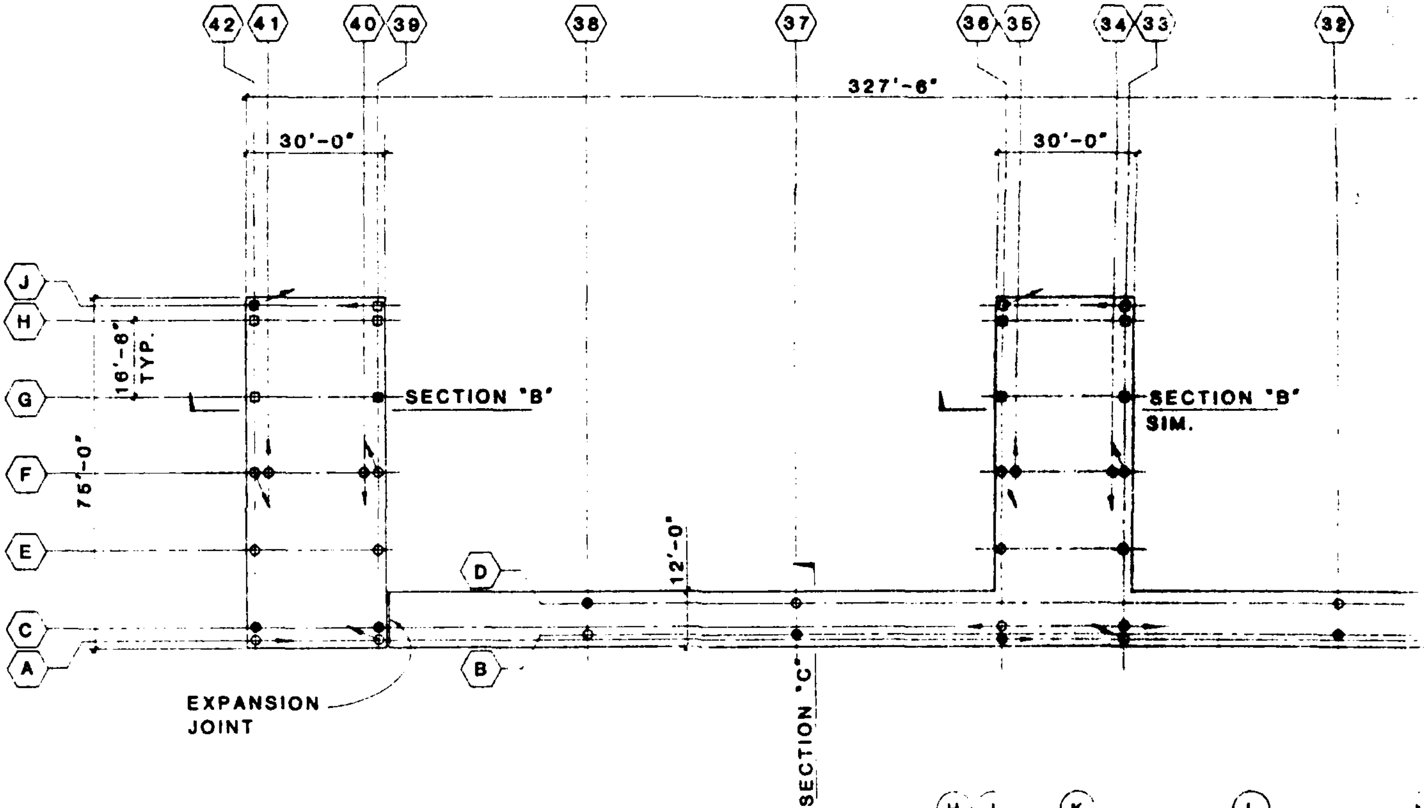
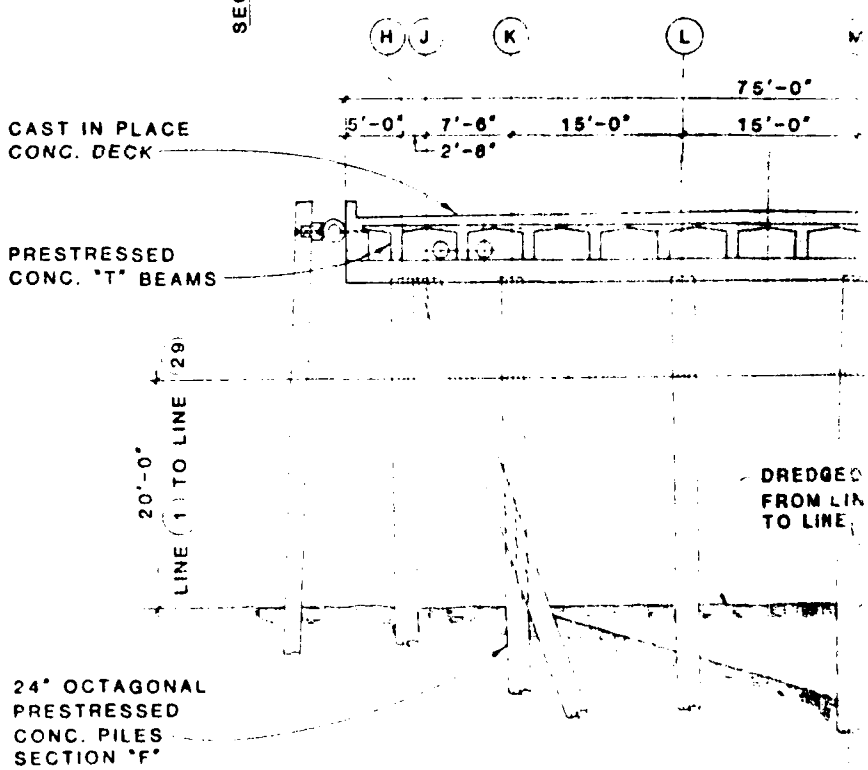
TAXIWAY NO. 1 SOUTH
TAXIWAY NO. 5 EAST
TAXIWAY NO. 6
TAXIWAY NO. 12
TAXIWAY NO. 3
WIND TEE
Parking Lot
SEAVIEW HERITAGE PARK
WEATHER STATION
pacific
ocean
North

FIG. 9

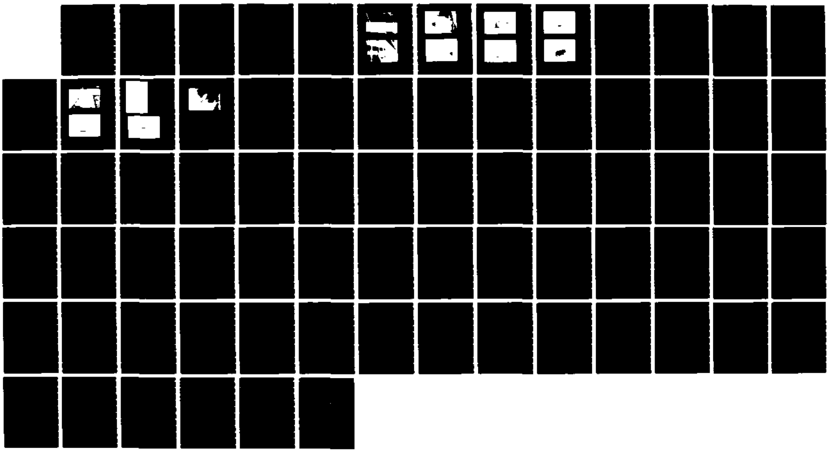


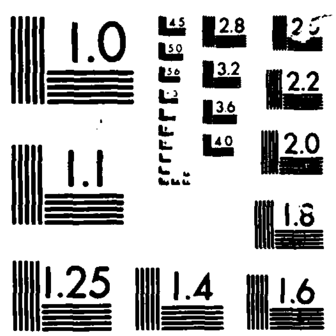


TYPICAL BENT - SEC
1/8":1'-0"

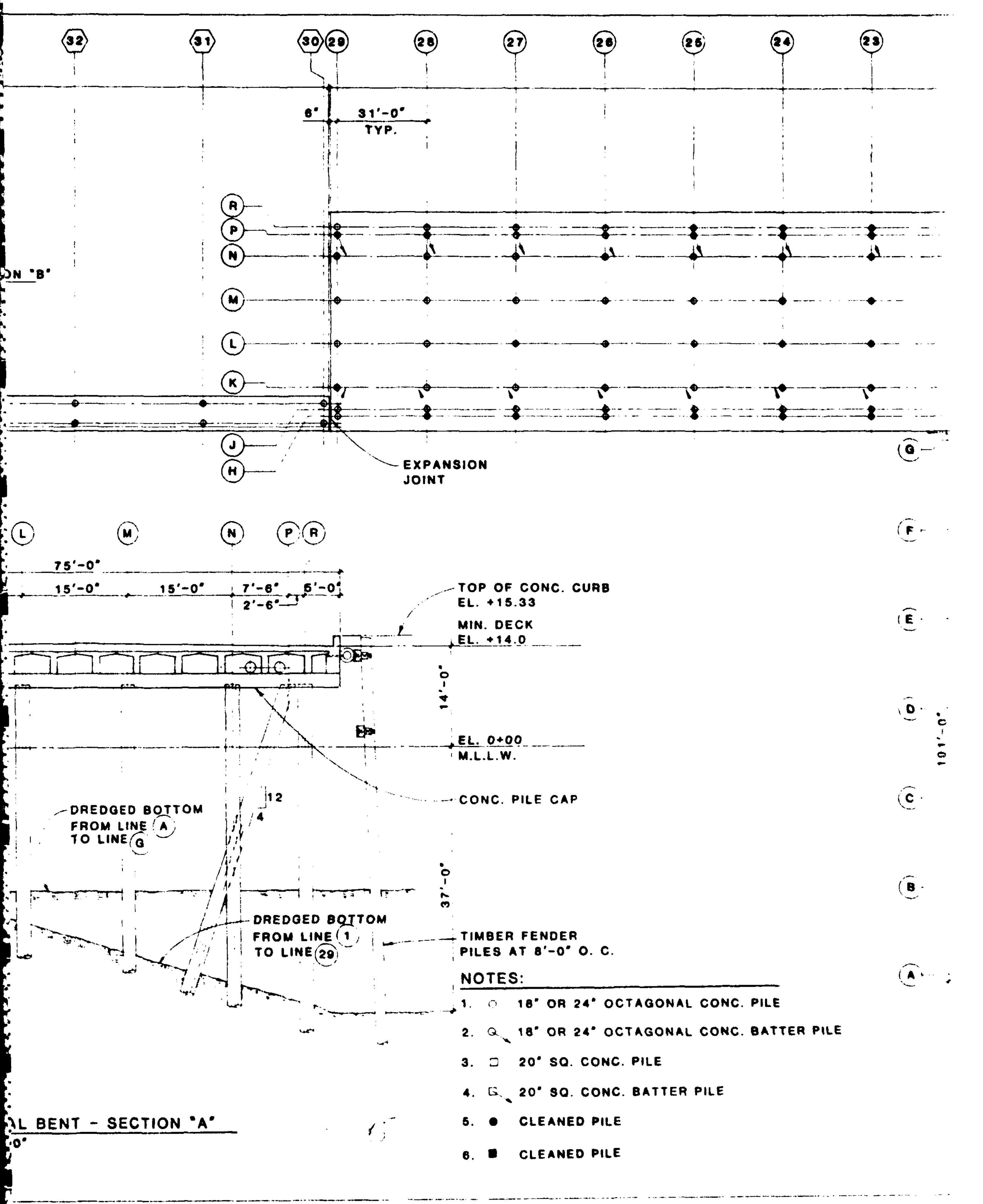
AD-A168 457 UNDERWATER FACILITIES INSPECTIONS AND ASSESSMENTS AT NAVAL AIR STATION NO..(U) BLAYLOCK-WILLIS AND ASSOCIATES SAN DIEGO CA NOV 84 2/2

UNCLASSIFIED CHES/NAVFAC-FPO-1-84(19) N62477-83-D-0190 F/G 13/2 NL





MICROCOP CHART


32
31
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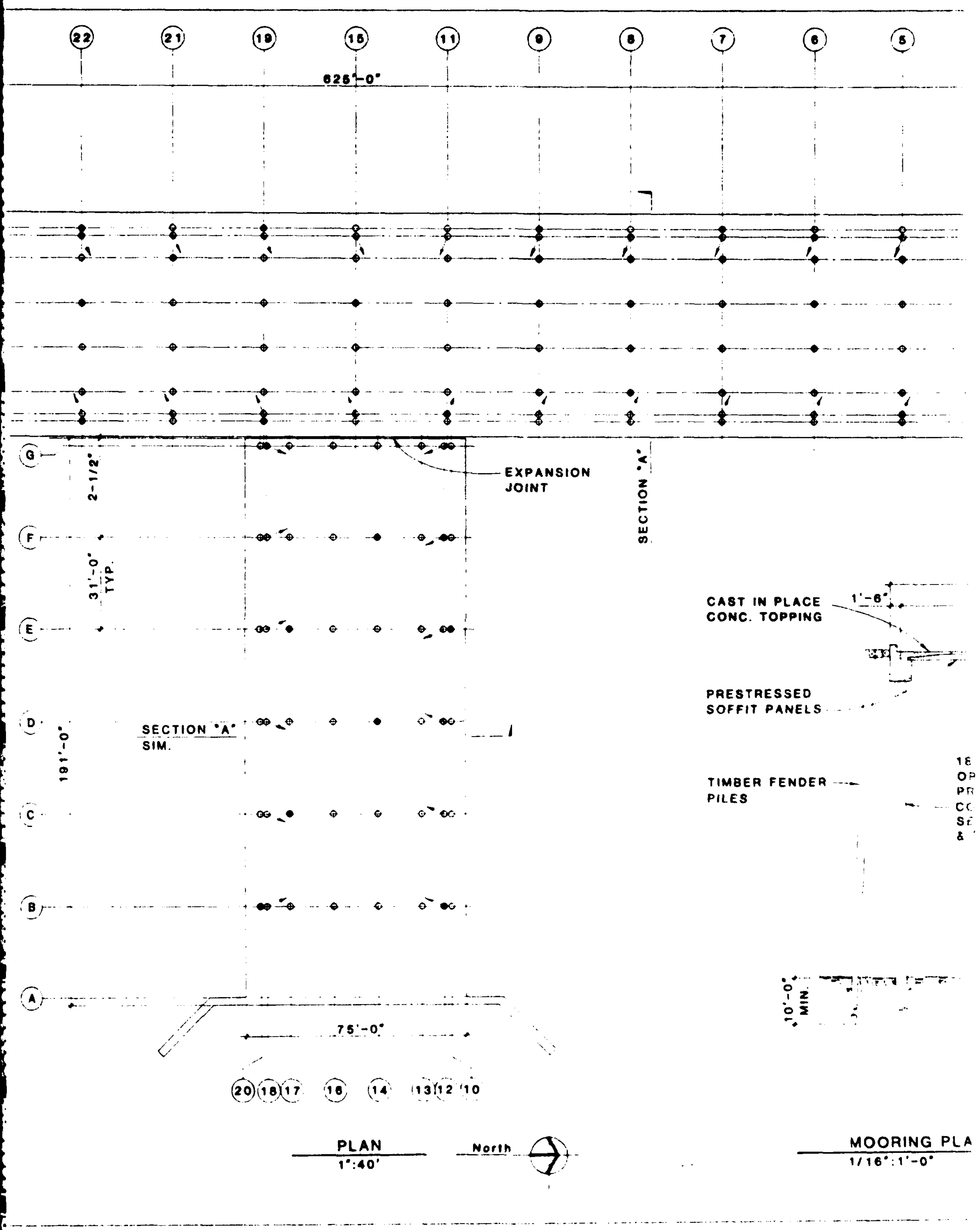
24
23
6"
31'-0"
TYP.
R
P
N
M
L
K
J
H
G
F
E
D
C
B
A
ON "B"
EXPANSION JOINT
75'-0"
15'-0"
15'-0"
7'-6"
5'-0"
2'-6"
TOP OF CONC. CURB
EL. +15.33
MIN. DECK
EL. +14.0
14'-0"
EL. 0+00
M.L.L.W.
101'-0"
CONC. PILE CAP
12
4
DREDGED BOTTOM
FROM LINE A
TO LINE G
37'-0"
DREDGED BOTTOM
FROM LINE 1
TO LINE 29
TIMBER FENDER
PILES AT 8'-0" O. C.
NOTES:
1. 18" OR 24" OCTAGONAL CONC. PILE
2. 18" OR 24" OCTAGONAL CONC. BATTER PILE
3. 20" SQ. CONC. PILE
4. 20" SQ. CONC. BATTER PILE
5. CLEANED PILE
6. CLEANED PILE
L BENT - SECTION "A"
0"


22
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625'-0"
G
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D
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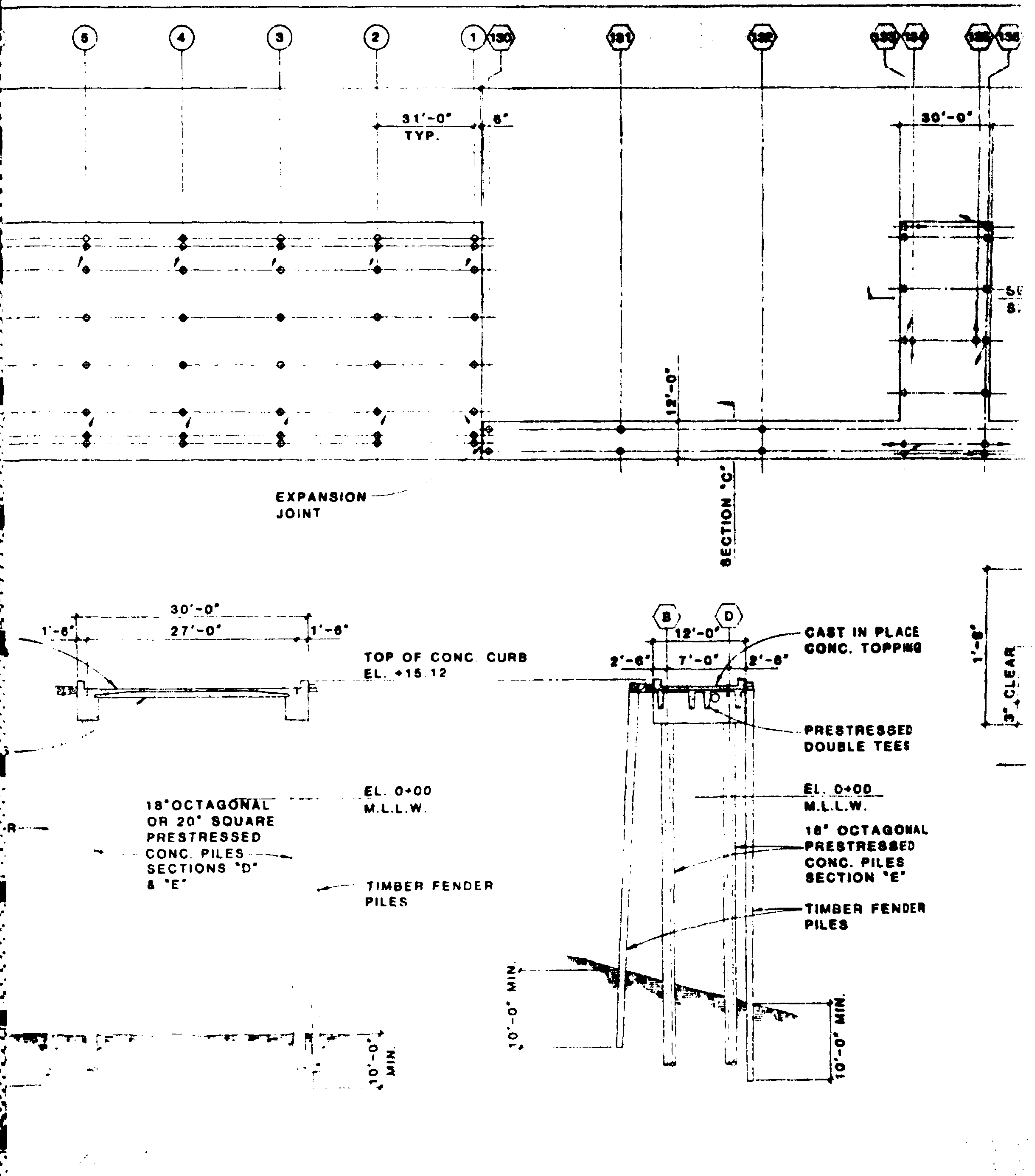
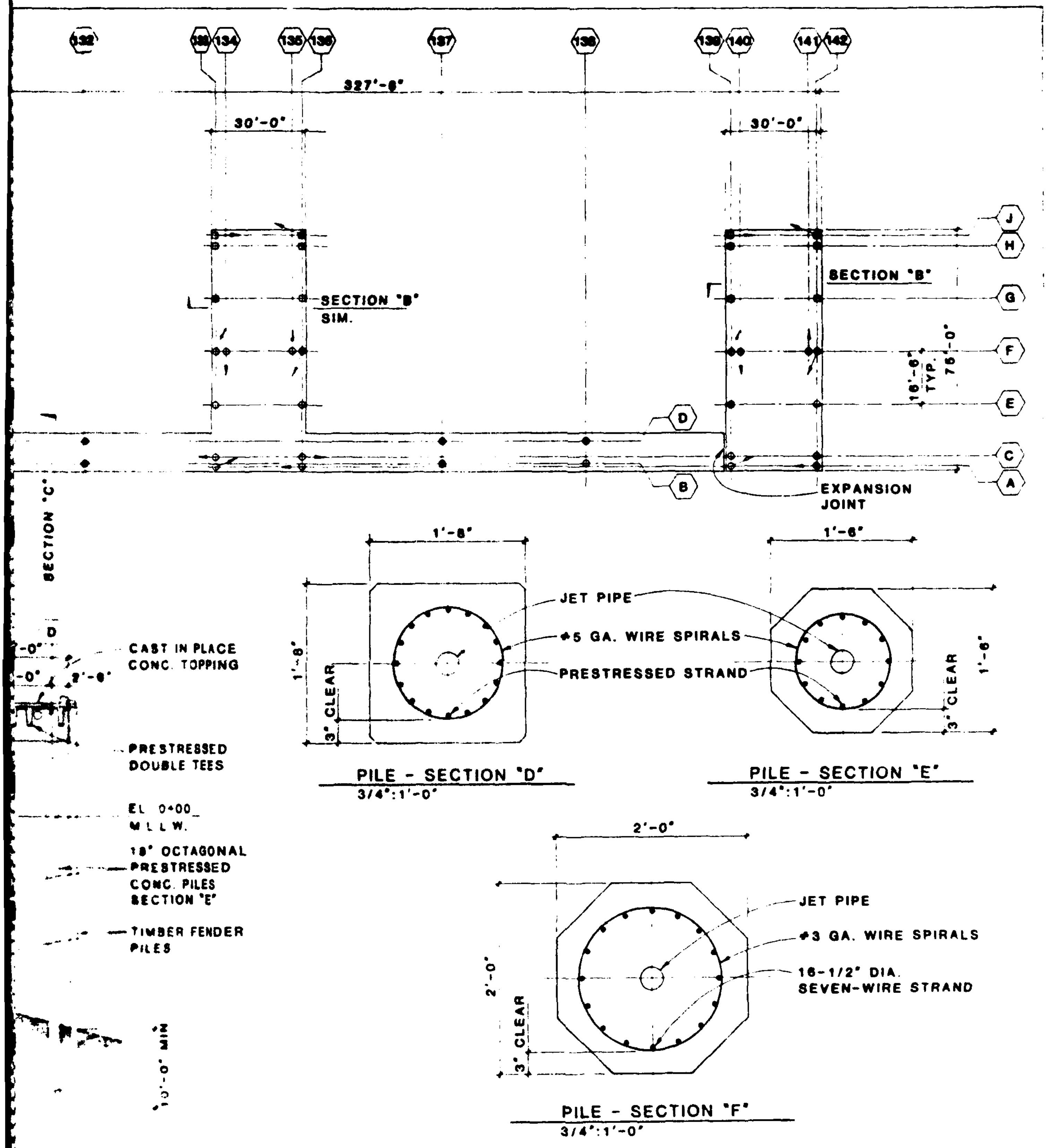
2-1/2"
31'-0" TYP.
191'-0"
SECTION "A" SIM.
EXPANSION JOINT
SECTION "A"
CAST IN PLACE CONC. TOPPING
1'-6"
PRESTRESSED SOFFIT PANELS
TIMBER FENDER PILES
10'-0" MIN.
75'-0"
20
18
17
16
14
13
12
10
PLAN
1":40'
North
MOORING PLA
1/16":1'-0"



MOORING PLATFORM - SECTION "B"
1/16":1'-0"

WALKWAY - SECTION "C"
1/16":1'-0"



PIER BRAVO
PLAN AND TYPICAL SECTIONS
NAVAL AIR STATION - NORTH ISLAND, SAN DIEGO, CALIFORNIA

Blaylock-Willis and Associates
STRUCTURAL ENGINEERS SAN DIEGO, CALIFORNIA

DATE: OCT. 1984

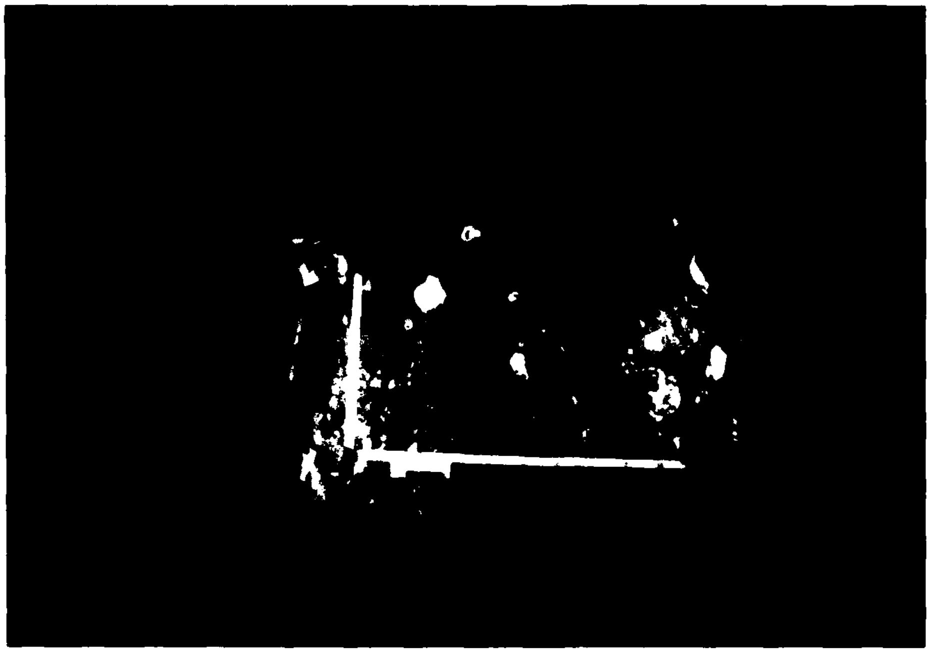
FIG. 10

32. Pier Bravo, view is to the northwest. The north extension, built 1979, begins at the change in pattern of the fender system.

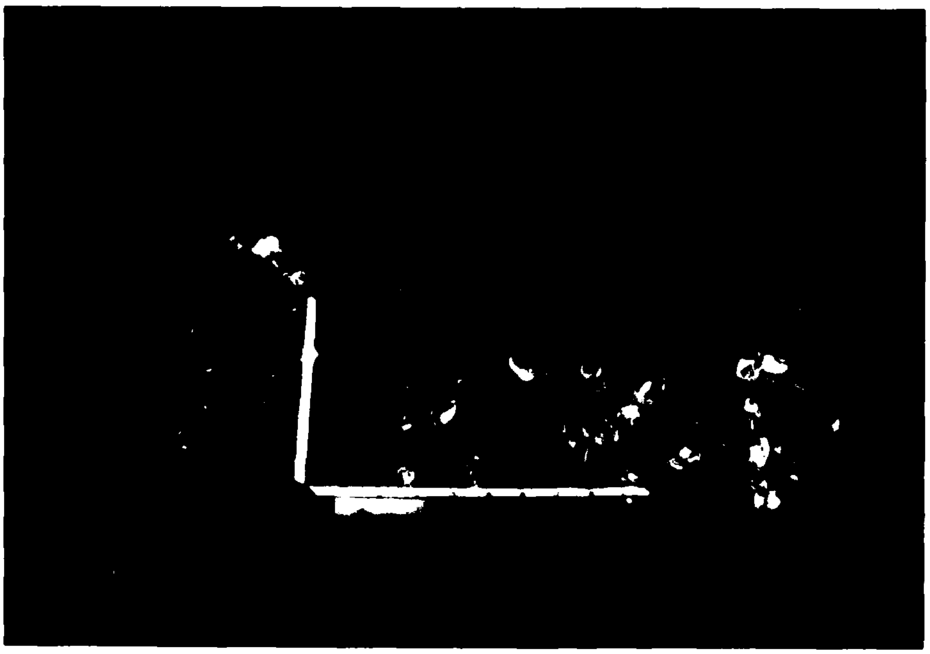
33. Pier Bravo, view is to the north, of the north extension of the pier.

34. Pier Bravo, view is south of the underside of the original pier. Its structural condition is excellent.

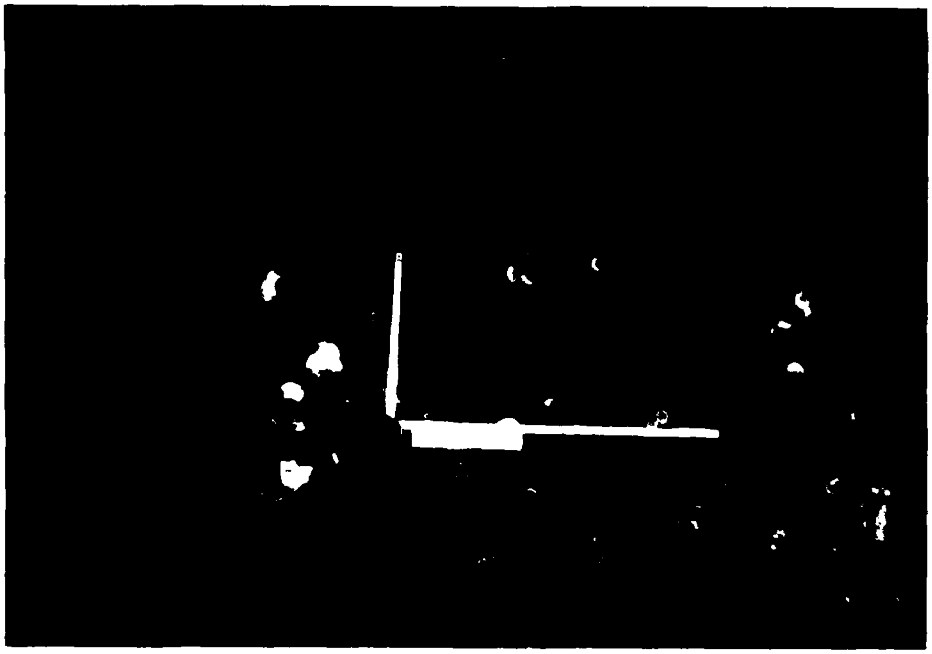
35. Pier Bravo, Diver cleaning Pile 136-J near bottom. The marine growth is very heavy on Pier Bravo.



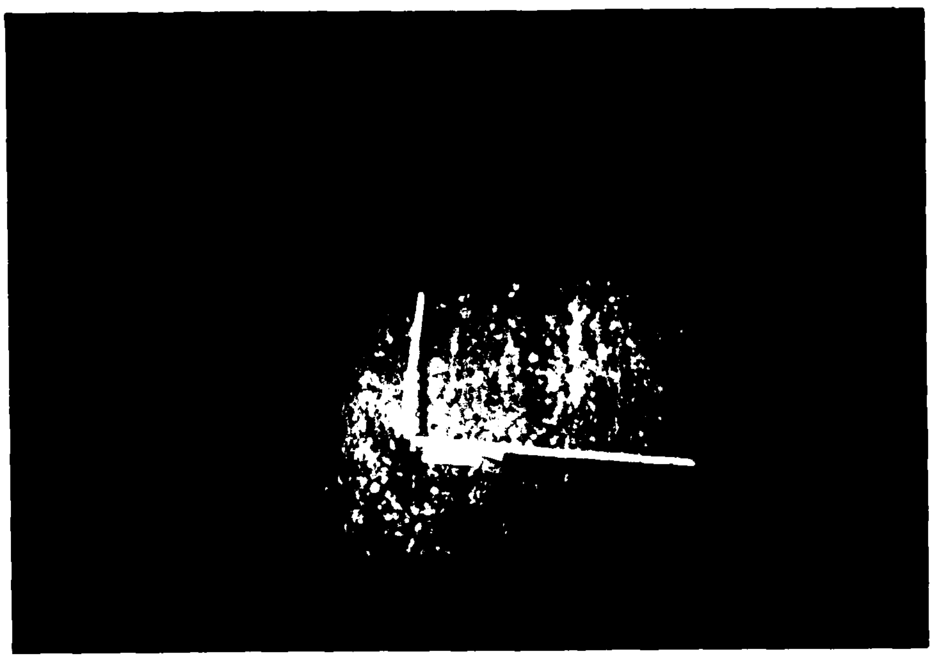
36. Pier Bravo, Pile 2-N at cleaned band at bottom. Pile is 24" solid octagonal unit. There is no evidence of any structural pile damage at this pier. Striking pile with pointed hammer produces spall to small to photograph.



37. Pier Bravo, Pile 133-G at cleaned band at mid-height. Pile is 20" square.



38. Pier Bravo, Pile 136-J at cleaned band mid-height. Pile is 20" square.



39. Pier Bravo, Pile 131-D at cleaned band at top. Pile is 18" octagonal.

4.4 PIER J/K

4.4.1 DESCRIPTION OF THE FACILITY

The object of this inspection of Pier J/K was to investigate and reconsider the assessments and recommendations of an extensive underwater investigation conducted in the summer of 1981.

The Pier was constructed in three increments in 1921, 1930 and 1958. The 1981 report entitles these increments as Regions I, II, and III respectively. Considerable sulphate damage has occurred to the concrete and reinforcing steel of Regions I and II. It is reasoned that this damage has resulted from the use of Type I cement which in current practice is not considered suitable for saltwater use. The more suitable Type II cement was not readily available on the West Coast until some time after 1940. The concrete in Region III is in good condition. A letter report confirming the conclusions of this investigation was issued August 10, 1984.

4.4.2 OBSERVED CONDITIONS

This inspection was conducted with the usual chipping hammer and bar scraper being used to clean and test the piles aided by equipment provided by a team of scientists from NCEL Port Hueneme. Of service in cleaning the piles were a water operated rotary brush and high pressure water jet unit. In addition an underwater modified Schmidt hammer was used to measure surface hardness, an R meter or pachometer also modified for underwater use was used to test for reinforcement location and an ultrasonic device developed for assessing concrete strengths underwater was tested.

Approximately 35 piles were cleaned and tested in the present inspection and found to confirm the assessment of the 1981 inspection.

4.4.3 STRUCTURAL CONDITION ASSESSMENT

In 1973, plans were prepared to update the utilities and to provide structural repairs to the above-water sections of Pier J/K. The structural repairs consisted of removing or cleaning rusted reinforcing bars and replacing lost concrete cover with gunite. This type of repair is expected to provide a service life in the vicinity of 15 years. That design contract did not include underwater investigation.

A subsequent study prepared in 1977, recommended that the pier be posted for a live load of 100 pounds per square foot, and the limitation of crane loading to that imposed by a 15 ton crane. Again, the study did not include underwater investigation.

The evidence of concrete pile deterioration observed in the 1981 investigation and confirmed in the present investigation, is sufficient to indicate a considerable loss of strength in the pile concrete. The deterioration is evidenced by surface softness. It is not uniform, but rather varies from place to place and between different locations on the same pile.

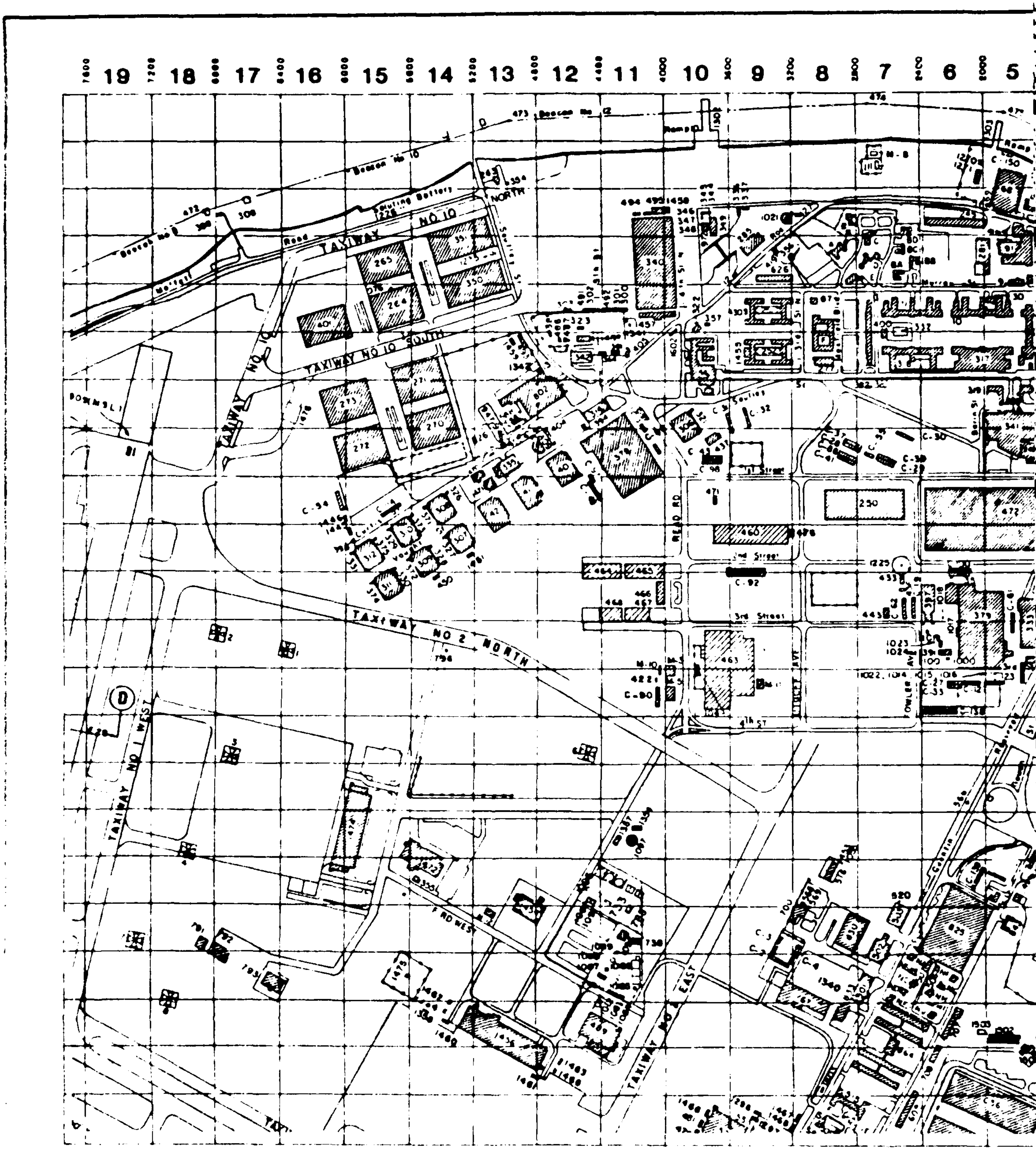
Photographs 42 and 43 show an extreme example of deterioration of a Region II pile. Photograph 41 shows one of the 1958 prestressed concrete piles which are in excellent condition. The concrete deterioration observed in Region I and II piles is due principally to the chemical reaction between the aggressive sulfate ion in seawater, and both the tricalcium aluminate and the calcium hydroxide in the Type I cement. The tricalcium aluminate reaction

results in the production of ettringite and the softening observed. In the worst cases (photographs 42 and 43), the depth of deterioration is in excess of 3 inches. Region I piles were originally 14 inches square and Region II piles were originally 16 inches square. (See Section 5.2, "Discussion of Pertinent Chemical Damage"). Section 4.4, "Structural Review of Piling", of the 1981 report indicates a typical damaged Region 1 pile to have 45% of its original design capacity without considering slenderness reduction. It indicates the typically damaged 16" pile to have 53% of its original capacity without further reduction for slenderness. However, in the more heavily damaged piling, the extent of softness is beyond the plane of steel reinforcement, so that these bars are separated from the competent core concrete. Compression buckling of a brittle material column so constructed is very complex. Obviously, these damaged piles do not conform with the assumption implicit in the design of reinforced concrete columns.

4.4.4 RECOMMENDATIONS

The result of the present inspection is to reaffirm the recommendations of the 1981 investigation.

Those recommendations included the limiting of live loads on the pier to 100 pounds per square foot, and the limitation of crane loads to those imposed by a 15 ton crane. This loading criteria was intended to apply for only five years, or until the summer of 1986. The best engineering judgement of the writer directs that the loads thereafter should be limited to that imposed by small vehicles and small craft berthing, with no crane loading.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Beacon No 10
Beacon No 12
Saluting Battery
TAXIWAY NO 10
TAXIWAY NO 10 SOUTH
NORTH
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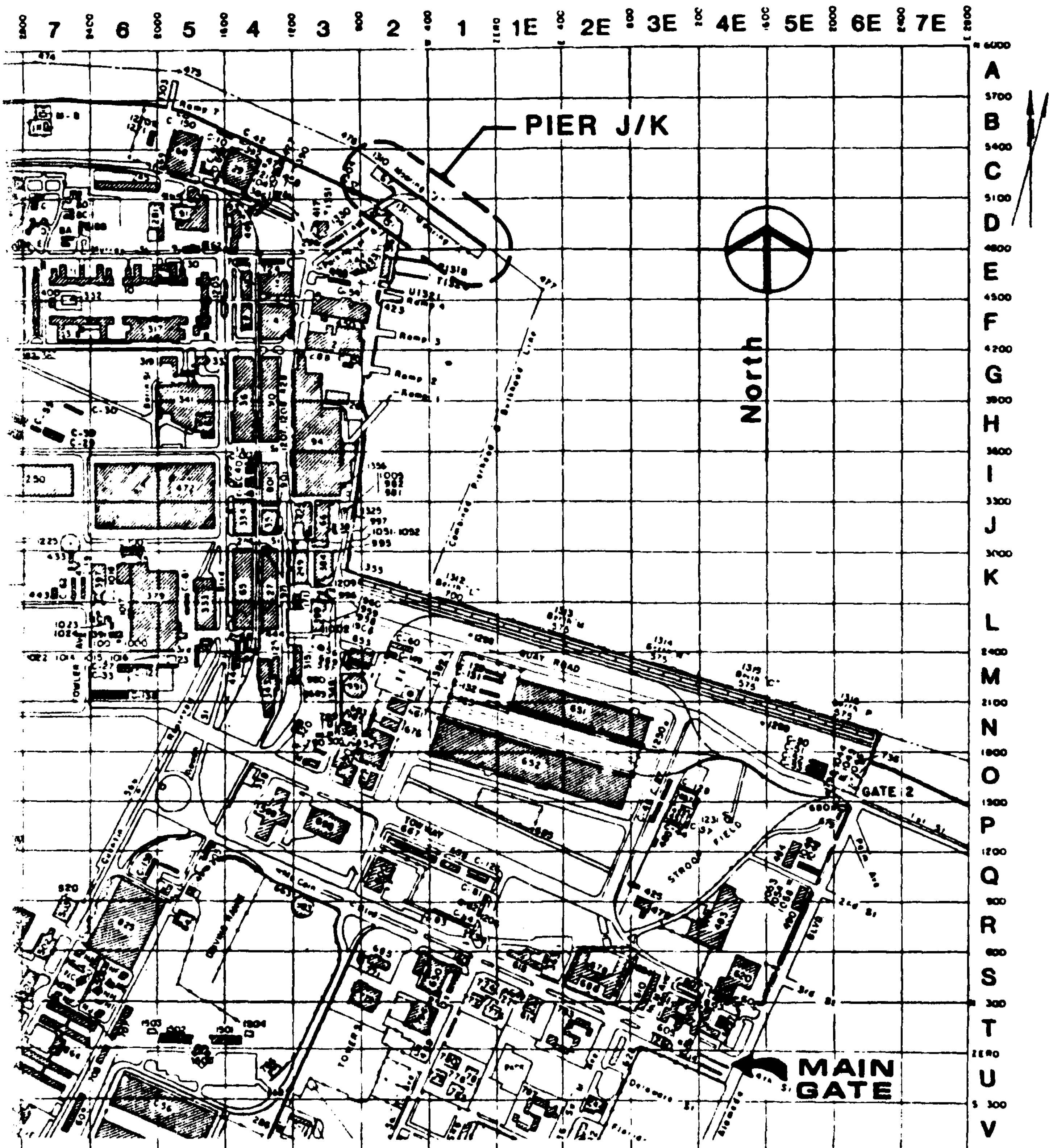
TAXIWAY NO 1 WEST
READ RD
2nd Street
3rd Street
4th ST
D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J
K
L
M
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O
P
Q
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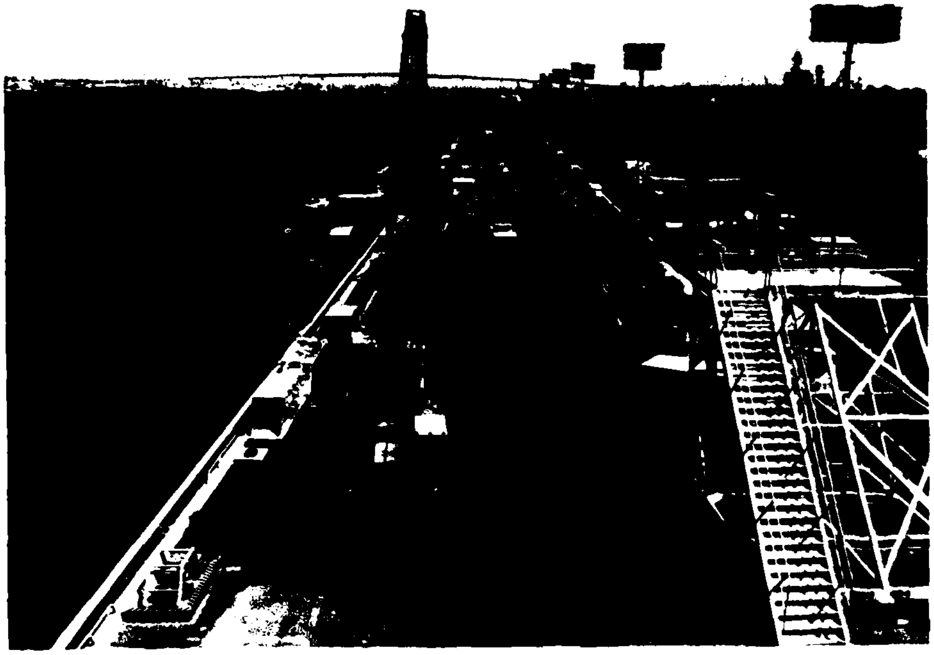
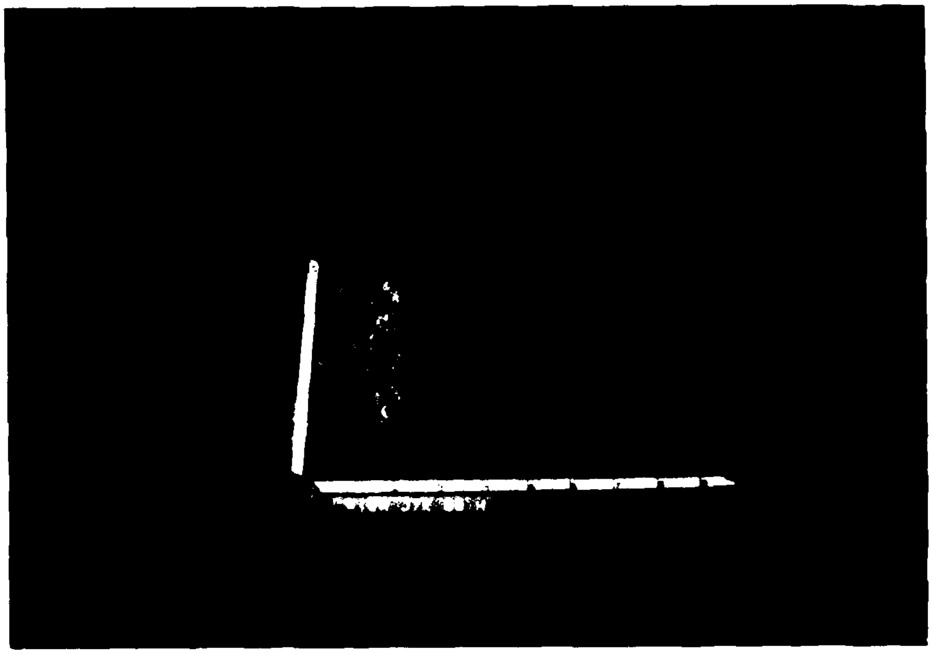
S
T
U
V
PIER J/K
North
QUAY ROAD
GATE 2
MAIN GATE

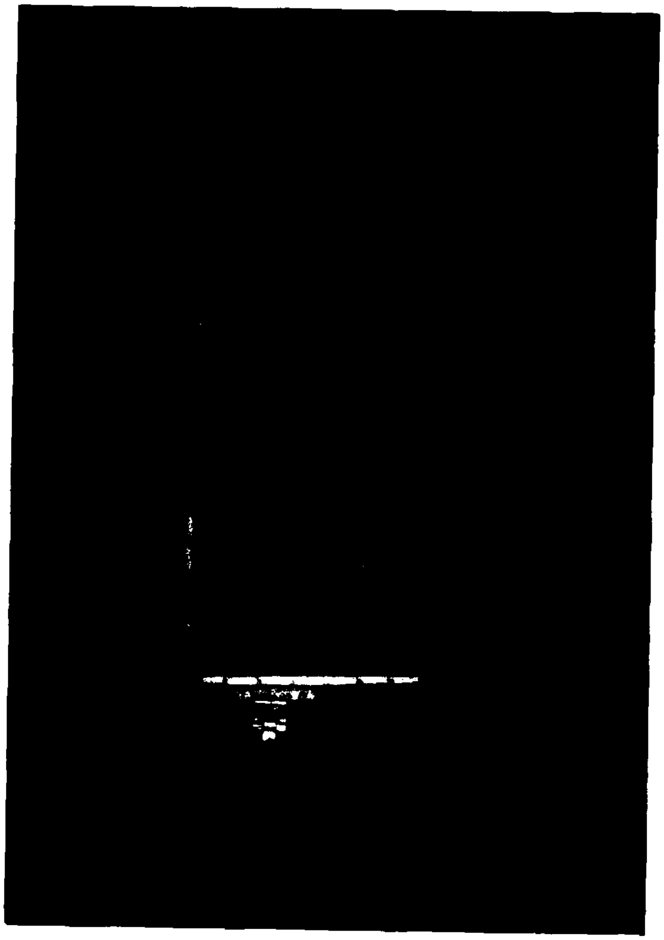
FIG. 11



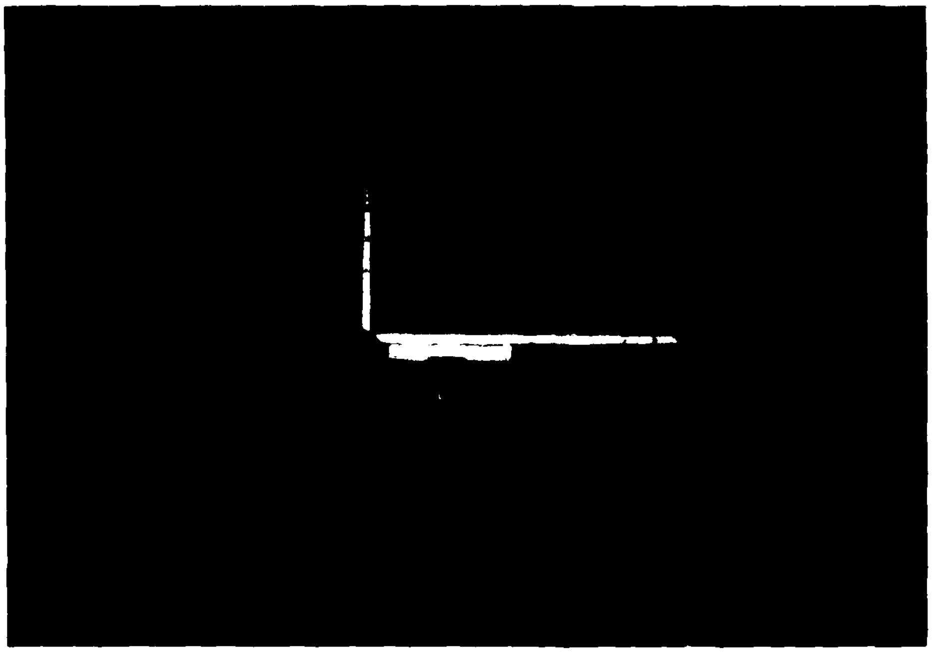
40. Pier J/K, photograph taken from derrick platform. View is to the southwest.



41. Pier J/K, Pile 88M at mid-height. This Pile is in Region III (1958 addition). It is in very good condition exhibiting no surface softness. Unfortunately Region III represents only 12 percent of the total pier area.



42. Pier J/K, Pile 9H mid-height. Region II (1930 addition) 16 inch square pile. The red bar was exposed in 1981 investigation. The blue bar was exposed in present investigation by use of water blast equipment.



43. Pier J/K, Pier 9H. The red bar is a #8 corner bar and the blue bar one of six #7 bars in the cross section.

44. Pier J/K. Diver is cleaning pile with experimental water jet device being perfected, by NCEL, Port Hueneme.

SECTION 5 - APPENDICES

5.1 PERSONNEL ON PROJECT

1. Chesapeake Division Personnel:
   Phillip Scola - Program Manager
   Wade Casey - EIC
   Christopher Crilley - EIT

2. Blaylock-Willis and Associates Personnel:
   A.J. Blaylock - Civil/Structural Engineer, Diver
   James Willis - Civil/Structural Engineer, Diver
   Daniel McNaughton - Civil/Structural Engineer, Diver
   Matthew Martinez - Civil Engineer, Diver
   Carson Creecy - Civil Engineer, Diver
   Thomas Spencer - Civil Engineer, Diver
   Darrell Williams - Structural Technician -Tender

3. Testing Engineers Inc. Personnel:
   Tony Rychell - Ultrasonic Equipment Technician

4. Studio B Photography Personnel:
   Lee Peterson - Underwater Photographer

5.2 DISCUSSION OF PERTINENT CHEMICAL DAMAGE

This discussion is predicated on information found in the attached Bibliography; in discussions with Peter Hawkins, Chemical Engineer of the California Portland Cement Company; Robert Tobin, Civil Engineer of the Portland Cement Association; and the experience of many years of observation of concrete structures and their problems by the writer.

The degree of deterioration of the various piling observed by the Blaylock-Willis and Associates' divers during the course of this investigation varies greatly between the facilities inspected. The damage to the piling of Pier J/K and the adjacent Air Station Bulkhead is sufficient to indicate a considerable loss of strength in the members. The damage to the sheet piles of the Carrier Bulkhead is not sufficient to significantly effect their strength. No damage to the piles of Pier Bravo was observed.

At Pier J/K and the Air Station Bulkhead the concrete in many places is soft enough to be chipped away with an archeologists pick revealing an unusually white color.

Typically, sulphate erosion of Portland cement - concrete begins with a subtle softening of the surface material. With time and continued exposure to the sulphate ion, the damage progresses deeper. In the later stages of the resulting decay, the surface very often becomes raveled as surface materials fall away. Chipping away of the softened material exposes a surface unusually white in color. In general, where a gray matrix of cement water paste is expected, a white material with the appearance of gypsum is exposed. The exact nature of the softening process is very involved and beyond the scope of discussion. "The mechanism of concrete corrision is extremely complex for it depends on a certain number of parameters which are not always easy to isolate and which react in

varying degrees according to the composition and the exposure of the material."[3] However, the general nature of the principal damaging reactions are quite commonly known and fit the evidence found at the Naval Air Station.

The literature makes reference to the aggressive ions in salt water. This reference is to the chlorine, the sulphate and the magnesium ions. The sulphate ion particularly has been the cause of much deterioration of concrete structures in the past.

One of the four principal compounds in cement is tricalcium aluminate ($3CaO.Al_2O_3$) which has the abbreviation "C_3A". Apparently, this compound is the principal target of attack for the sulphate ion, such that restricting the percentage of C_3A in the cement composition serves to reduce the amount of damage from sulphate attack. ". . .Sulphate-resisting Portland cement has a low content of tricalcium aluminate. . .which is the compound in Portland cement that is least able to resist chemical attack.

"The most promising cement of this type, as recommended by the U.S. Bureau of Reclamation, has less than 50 percent C_3A and less than 12 percent C_3A+C_4AF, in which less than 4 percent is C_3A. It has, after curing, a low content of hydrated lime. A requirement of ASTM Standard C150 for Type V cement is that the content of C_4AF plus twice the amount of C_3A shall not exceed 20 percent. The above-mentioned value of C_3A is based on chemical analysis rather than x-ray diffraction which gives a lower result. Portland cements approaching this composition, particularly with respect to C_3A, will give excellent service in cement-rich, densely compacted, properly cured concrete.

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(3) Bibliography, Page 65

"This cement is designed to resist attack by mineral sulphates in ground waters and subsoils, which contain sulphur trioxide in amounts up to 0.1 percent and 0.5 percent respectively. The characteristics of the cement. . .indicate it's suitable for use in aggressive environments such as heavily polluted or humid industrial atmospheres, sea water, factories and sewers. . ."[4]

Present-day Portland cements are divided into five types. The literature presents a somewhat confusing picture of the percentage of C_3A in the various cements. This is partly the result of the use of different means of determining this percentage, chemical analysis as opposed to x-ray diffraction. The Portland Cement Associations shows the C_3A percentage to be somewhat less than the limitations delineated in ASTM C150 or CSA A5. They describe these percentages as typical with the implication that the American Portland cements are well within the restricting criteria.[5] [6]

A long-term study resulting in two excellent references was conducted by the Portland Cement Association in which several hundred samples of various mixes of Portland cement were exposed to sulphate soils. The study was conducted over a period of twenty years and results in part in the following conclusion:

". . .conclusion No. 3[7] may now be supplemented to state that in addition, a C_3A content of about 5.5 percent as corrected for minor oxides and about 3.5 percent as determined by x-ray diffraction are fairly good values for separating superior and poor performance of the 7-bag concrete."[8]. . .in soils containing a high percentage of sulphate ion.

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(4) Bibliography, Page 19
(5) Bibliography, Page 16
(6) Bibliography, Page 28
(7) Bibliography, Page 33
(8) Bibliography, Page 14

Type I normal cement has approximately 11 percent of C_3A. Type II cement, which is considered to have some moderate sulphate attack-resistance capability, has 5 percent C_3A according to PCA's typical table and 8 percent according to ther ASTM C150 limitations. Type V sulphate-resisting cement has a C_3A content of 4 percent typically, and 5 percent the ASTM limitation.

Hawkins relates that only Type I was available on the West Coast of the United States through the 1930's. About 1940, Type II cement became available and approximately a decade later, or at least in the later 1940's, Type V cement became available.

When the sulphate damage is considerably progressed, the surface softening is accompanied by swelling and will cause cracks to occur at the corners of square piling. Sometimes these cracks will heal, becoming filled with a gypsum-like material, or with oxides of iron - hemotite (FE_2O_3) which is red, magnitite (FE_3O_4) which is gray-black, or ferrous oxide (FEO) also black and really incompletely oxidized iron. Also, some autogenous healing (deposition in the crack of efflorescence as the result of limestone ($CaCO_3$) production) may take place.

While the sulphate ion in its reaction with C_3A affects the concrete, the chlorine ion is principally responsible for the rusting of the steel as its presence ". . .causes the loss of passivity provided by the normal alkali protection of free lime in hydrated cement. . . ."[3]

The chemical product of the reaction between C_3A and the sulphate ion is called ettringite. Ettringite has the formula "$3CaO.AL_2O_3.31H_2O$." Obviously, with all the water of crystallization in its composition, the ettringite is a much larger crystal than the parent C_3A so the swelling, increased density and spalling seem like logical results of the suphate attack. Unfortunately, softening is also a result.

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(3) Bibliography, Page 64

There are descriptions in the literature regarding sulphate deterioration which are not entirely consistent with the evidence observed at the Naval Station. "The deterioration of marine structures constructed with mortar or concrete are of chemical and physical nature. If the structure is fully immersed, the attack on the material by sea water is essentially chemical. It is related to the dissolution of lime and to the expansive formation of ettringite which lead to erosion, swelling, cracking and spalling. In alternating immersion and exposure conditions, the attack is of chemical and physical nature. The mechanical action of the waves, the swelling and shrinkage caused by the alternate saturation and drying, atmospheric conditions (wind, exposure to the sun, freezing) and the electro-chemical corrosion of steel reinforcement are physical processes which add to the chemical destruction processes."[3]

The above reference would lead to the expectation that the sulphate damage would be greater in the tidal range. It speaks of the mechanical action of the waves and swelling and shrinkage due to saturation and drying. The inspection of these piles indicated very clearly that concrete material in the region below low water exhibited the greater damage, that indeed the tidal range concrete was quite sound. Although this evidence may not be consistent with the reference, we submit that the greatest opportunity for sulphate damage to be progressed should be favored by the material in the water for the longest period of time.

The matter of gypsum occurring in the concrete is of some interest. The literature suggests that another sulphate reaction with the cement is possible. That is the formulation of gypsum ($C_aSO_4.2H_2O$) from calcium hydroxide which is $Ca(OH)_2$. To begin with, there is a small amount of gypsum already present in the unhydrated cement.

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(3) Bibliography, Page 64

"The final grinding is done with a 3 to 5 percent addition of gypsum, which retards the hydration of the aluminate component of the cement and renders it fit for use; raw cement otherwise would have a tendency to flash set on the addition of water."[4]

The amount of white substance found in the softened areas of the piling at the Naval Air Station was considerably greater than the small percentage described above. The best evidence supports that presence of ettringite and gypsum with other chemical reactions also taking place. All of our references point out the fact that the nature of chemical activity between salt water and the cement particle is very complex with the principal culprit being the sulphate ion.

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(4) Bibliography, Page 6

5.3 DISCUSSION OF PERTINENT BIOLOGICAL DAMAGE

Reports of underwater inspections of concrete structures have suggested the presence of damage due to water-borne biological species. Any considerable discussion of this subject is considered beyond the scope of this report. However, a few comments regarding the various phyla present at the facilities investigated in this report, limited by the circumscribed knowledge of the observer - of salt water biology - would not be out of order.

The only recognizable animal observed to be occupying the interior of concrete columns was a species of Polychaeta which we would guess to be the Pareurythoe Californica.[9] It was revealed on several occasions while chipping at the corners of piles in Regions I and II. (Underwater Facilities Inspections & Assessments, Piers, 1, 3, 4, 5, 6, & 8, Naval Station, San Diego, California, FPO-1-82-(09), November 1981.) It is reasoned that the swelling of the surface of the columns is due to the growth of ettringite which causes the corners to spall away from the cross-section to a depth of an inch or two. The resulting crack - at 45 degrees from each surface at the corner and vertical - provides habitat for the crack dwellers including the Polychaeta. But then this hardly constitutes damage from bio species. The damaging crack occurred before the worm occupied it.

No species resembling Pholadidae [9] were observed or were burrows or holes purported to be the result of their presence observed.

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(9) Bibliography, Page P138

(9) Bibliography, Page P126

5.4 ASCE PAPER, "CAVE-INS OF SANDY BACKFILLS"

BY J. C. GEBHARD, M. ASCE.

CONTENTS FOR JANUARY, 1948 (*Continued*)

GUIDEPOST FOR TECHNICAL READERS
JANUARY, 1948, *PROCEEDINGS*

Items of value or significance to readers in various fields are here listed. For convenience, the distribution is given in terms of the Society's Technical Divisions

(A constant effort is made to supply technical material to Society members, over the entire range of possible interest. In so far as your specialty may be covered inadequately in the foregoing list, this fact is a gage of the need for your help toward improvement.—Ed.)

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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

LATERAL EARTH PRESSURES ON FLEXIBLE RETAINING WALLS

A SYMPOSIUM

NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by June 1, 1948.

INTRODUCTION

BY WILLIAM H. SMITH,[1] M. ASCE

The Bureau of Yards and Docks, United States Navy Department, has always been deeply involved in the problems of heavy earthwork. Much of the Navy's shore establishment is located on existing harbors and waterways, where the engineer must accept foundation and subsoil conditions as he finds them. Almost any harbor improvement project involves filling out beyond the natural shore line, building a quay wall or bulkhead, and dredging outside it. In many cases the engineering problems of the Navy are rendered more difficult because of the heavy surcharges to be provided for. The weights of guns, barbettes, turrets, armor, and other components of capital ships, the generally heavy character of equipment, and the tendency to do more and more prefabrication and preassembly--all lead to heavier types of shore construction.

On the other hand, rising costs and shortages of materials lead to progressive development of designs to utilize modern theories, materials, and methods to the best possible advantage. The days of massive, granite-faced gravity walls, for example, seem to have passed. These developments impose greater and greater responsibilities on engineers to accomplish economical results without reducing the factor of safety inherent in their structures.

The best way to improve engineering design is to improve knowledge of the forces to be resisted and of the mechanics of the structures in resisting them, so that every pound of material is working at the fullest possible efficiency and yet is never overstressed. The test program undertaken in 1943 at Princeton University (Princeton, N. J.) under the sponsorship of the Bureau of Yards and Docks has this specific objective in view. By careful laboratory studies of lateral earth pressures, and, particularly, studies of methods to reduce the lateral fluid pressure of hydraulically placed backfills, engineers can insure against failures of bulkheads and can arrive at more economical designs that will be safe and reliable. By further study engineers can learn even more about the behavior of flexible bulkheads, and the effect of bulkhead displacement on *soil pressures, so that by modifying designs or methods of construction or both,* they can build better bulkheads at lesser cost.

Briefly, the broad topics studied under this program can be divided into four main categories. First, investigation was launched into the determination of lateral pressures exerted on flexible steel bulkheads by backfills of clay, a sand-clay mixture, and a clean sand. This phase of the problem was important in order to obtain the basic information as to the lateral pressures to be expected with backfills of these types. The second phase of the problem was to determine methods for reducing these pressures. Tests were conducted to determine the effectiveness of sand dikes, placed on a normal one-on-two slope behind the bulkhead, in reducing lateral pressures. The effectiveness of the sand dike and of other methods attempted will be covered in the first Symposium paper. As the third phase of the investigation, and as a corollary to the sand-dike experiment, tests were made to determine the effectiveness of a sand blanket of varying widths placed back of the bulkhead in reducing lateral pressures. A fourth investigation was launched into the development of a method of accelerated consolidation of clay backfills designed to reduce quickly the excess pore pressures in the backfill and, correspondingly, to reduce the lateral pressure on the bulkhead. Finally, as a result of these studies, data of basic importance have been obtained for the analysis of the phenomena related to the problems of lateral pressures of soils, and many avenues of approach for further research and study in this field have been revealed. The present status of the tests, the preliminary findings and conclusions, and the contemplated future course of action on the completion of the test program are fully discussed in the papers.

[1] Rear Admiral, CEC, U. S. Navy, Washington, D. C.

Several aspects of the Symposium deserve emphasis:

(1) It is not a final statement of results. It is rather a progress report on what has been accomplished, a discussion of the tentative conclusions and interpretations of the test results, and an indication of the direction that further tests will take.

(2) Although there have been free and intensive discussions among Professor Tschebotarioff, his associates at Princeton, and the designing engineers of the Bureau of Yards and Docks, the Navy has not attempted to force any unanimity of conclusions or opinions. It is only by the application of basic principles of free speech--by the full and free presentation of divergent points of view—that a meeting of minds and a constructive, intelligent, and sound set of principles can be obtained.

(3) It is neither the intention nor the hope of the sponsors of this investigation to resolve, once and for all, the many open questions in the field of soil mechanics. In one who does not attempt to qualify as an expert specialist in this science, there frequently arises a feeling of confusion and futility when one tries to reconcile the deep and apparently fundamental disagreements between different authorities on the interpretation and application of the theories of soil mechanics.

The general practitioner in civil engineering must make allowances for the relative youth of this branch of engineering as a science. He must recognize the complexity and variability of the materials with which these experts deal, and must have patience with them in their efforts to bring law and order into an involved and intractable situation.

Medical doctors have been struggling for years to solve the mystery of the common cold. Perhaps, in comparison, the engineering profession should applaud soils engineers for the great progress they have made in clarifying and delineating the behavior of mother earth, rather than look askance at their achievements because they are occasionally clouded by controversy. These active differences must be recognized rather, as proof in themselves of the virility, the vigor, and the dynamic character of this young science.

NOTATION

The relatively few letter symbols in this Symposium conform essentially to "Soil Mechanics Nomenclature," ASCE *Manual of Engineering Practice* No. 22. Discussers are requested to adapt their symbols to this list:

A = reaction at level A;
A' = the slope of the semilogarithmic plot of a void-ratio curve (Eq. 30); $A = A'/2.306$;
B = reaction at level B; also B = a constant in Eq. 30;
C = reaction at level C;
C' = a constant defined by Eq. 9;
c = cohesion per unit area:
D = diameter; average diameter of a pile;
E = modulus of elasticity in bending and tension; E_G = modulus of elasticity in shear;
e = void ratio:
  e_{cr} = critical void ratio;
  e_o = void ratio at the surface, $h = 0$;
  e_1 = void ratio *corresponding to* p_c = *0.1 ton per sq ft;*
e = base of Naperian logarithms;
G = specific gravity of solids;
H = height of bulkhead:
  H'_c = critical depth, or the depth at which a vertical cut will stand unsupported;
  H_g = depth of ground water level below the ground surface;
h = vertical distance from the top of the bulkhead to any point *along* its vertical height:
  h_i = the depth where $(\gamma_s - \gamma_w)h_i$ is 0.1 ton per sq ft;
  h_o = distance h to any point along the vertical height of a bulkhead;
I = rectangular moment of inertia;
I_p = plasticity index;
K = hydrostatic pressure ratio; ratio of horizontal pressure to vertical pressure; a consolidated pressure coefficient:
  K_A = ratio for the reaction A at the anchor level A;
  K_a = ratio for active pressure;
  K_B = ratio for the reaction B at the anchor level B;
  K_h = ratio of pressures at depth h;
  K_p = ratio for passive pressure;
  K_R = ratio for the total pressure R;
  K_s = ratio for the buoyed weight of solids;
  K_{s+w} = ratio for the nonbuoyed combined weight of soil and water;
  K_ϵ = ratio for the bending strain, ϵ_o;
  K_1 = active pressure coefficient for a triangular sand dike in Fig. 40;
  K_2 = active pressure coefficient for a level sandbank;

L = span length; vertical distance between the A-level and the B-level of the model bulkhead; L_p = length of pile;
M = bending moment;
m = a factor defined by Eq. 29;
N = a ratio of K-values; N_o = the value of N for any height h_o;
n = tangent of the angle of wall friction;
P = pressure force exerted on a bulkhead (pounds per linear foot):
  P_a = total active force exerted on a bulkhead (pounds per linear foot);
  P_{af} = total active force exerted on a bulkhead, considering wall friction;
  P_{ax} = total active force exerted on a bulkhead for a variable angle, X, of rupture plane;
  P_{cr} = critical load on a pile;
  P_p = total passive force exerted on a bulkhead (pounds per linear foot);
  P_1 = total horizontal pressure due to weight W_1 (Figs. 43 and 45);
  P_2 = total horizontal pressure due to weight W_2 (Figs. 43 and 45);
  P_3 = total force from fluid clay (Figs. 43 and 45);
p = unit pressure (pounds per unit area):
  p_a = unit active pressure;
  p_c = capillary pressure;
  p_d = *difference in unit pressures;*
  p_h = unit lateral pressures;
  p_n = normal pressure;
  p_c = consolidation pressure;
  p_1 = the major principal stress; unit pressure for a triangular dike;
  p_2 = the minor principal stress; unit pressure for liquid clay;
  p_3 = unit pressure for a triangular dike;
  p_4 = unit pressure for liquid clay;
Q = ratio of friction angles n/μ;
q = unit compressive strength of unconfined specimens; q_c = unit strength of specimens consolidated under pressure corresponding to effective weight of overburden;
R = resultant force; total lateral pressure; horizontal reaction; subscripts indicate point of application;
S = ratio of pressures, P_{af}/P_a;
s = spacing distance; as a subscript, s denotes "solids"
t = a numerical factor less than 1.0 (berm width); ratio of top width to height of trapezoidal dike;
V = shear force, in pounds;
v = shearing resistance, shear force per unit area;
W = total value of a distributed weight w:
  W_1 = buoyed weight of sand dike above rupture plane (Figs. 43 and 45);
  W_2 = buoyed weight of fluid clay above rupture plane (Figs. 43 and 45);

w = a variable distributed load or weight; as a subscript, w denotes "water";
X = angle between the assumed rupture plane and the horizontal;
z = distance from the top of the bulkhead to the point of maximum bending moment on the bulkhead; z_o = distance from the top of the bulkhead to the point of theoretical zero pressure on the upper part of the bulkhead;
α = tangent of the angle X:
  α_a = tan X for active pressure;
  α_p = tan X for passive pressure;
  α_s = tan X;
β = tangent of the angle θ;
γ = weight per unit volume:
  γ_a = unit weight of soil in air;
  γ_e = effective unit weight of soil, as transmitted from grain to grain; equal to buoyed weight, when soil is submerged;
  γ_w' = unit weight of water;
  γ_1 = buoyed weight per unit volume for sand;
  γ_2 = buoyed weight per unit volume for clay;
ϵ = bending unit strain; ϵ_o = maximum strain of an equivalent beam freely supported at levels A and B;
μ = tangent of the angle ϕ;
ϕ = *angle of internal friction*; ϕ_c = *slope of a line relating unconfined compressive strength to consolidation pressure*; and
θ = surcharge angle (positive above level bank).

General Acknowledgment

The Bureau of Yards and Docks pays special tribute to the efforts that have been made, by Professor Tschebotarioff and his associates, in the preparation of this Symposium. Officers and designing engineers in the Department of Planning and Design, Bureau of Yards and Docks, also deserve hearty commendation. First, the duties imposed by World War II and subsequently the reduction of funds and personnel have added burdens on all of them. During the transition from a wartime to a peacetime footing it has not been possible to assign any Navy personnel to this task full time and all have had to do their share while handling many other assigned tasks.

PAGE 9 TO PAGE 83 NOT
RELEVANT TO THIS REPORT

CAVE-INS OF SANDY BACKFILLS

BY J. C. GEBHARD,[43] M. ASCE

SYNOPSIS

The other papers in this program have dealt principally with various laboratory and design aspects of lateral earth pressures against bulkheads. In applying experimental and theoretical knowledge to the building of actual structures, the engineer is never quite sure just how conservative his designs and construction procedures have been unless there is some visible sign of distress or actual failure. Since such occurrences help to build up the fund of practical judgment, Commander Coxe and the writer reasoned that it would be of interest to discuss a few jobs that gave disappointing results. This paper concerns a case involving a nonplastic sandy backfill whereas the last Symposium paper by Commander Coxe describes a job that involved plastic soils.

DESCRIPTION

During World War II, it was necessary to increase the land area at one of the United States Naval stations by filling an indentation in the harbor with hydraulically placed sandy material. It was also necessary to construct a

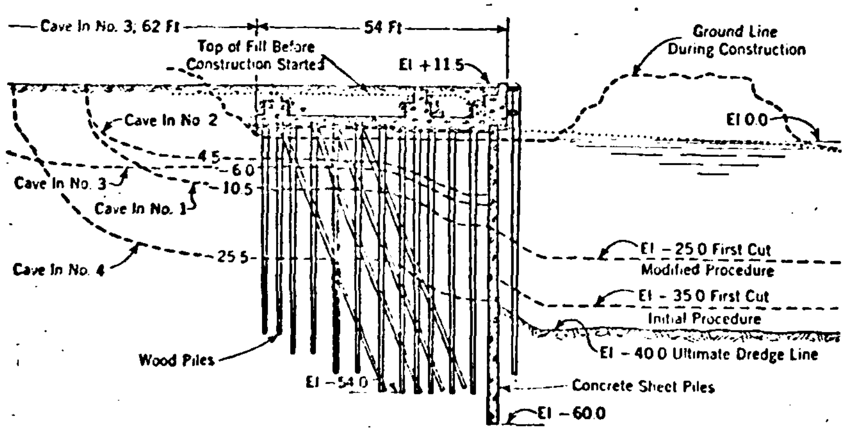

FIG. 53.—TYPICAL CROSS SECTION THROUGH QUAY WALL

3,000-ft quay wall across the outer end of the bight to retain the fill and to provide a wharf with a 40-ft depth of water alongside.

The quay wall (see Fig. 53) is of the relieving platform type with concrete sheet piles on the outer face. A flat slab concrete platform, about 53 ft wide,

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with its bottom surface about 1.5 ft above low water is supported on a mat of closely spaced untreated wooden piles (see Fig. 54). The concrete sheet piles are anchored to the platform and the lateral thrusts are taken by wooden batter piles. A gravity wall extending to El. 11.5 forms the outer edge of the wharf and retains the surface fill.

The depth to the original sandy bottom varied considerably but was taken as El. −20 for purposes of design. Before starting work on the quay wall, a wide sand dike was constructed on the line of the quay wall to about El. + 11. The dike material was placed by a slotted dredge discharge pipe and it is reported that most of the thin overburden of muck or silt on the bottom of the bay was washed out in the process. Wooden test piles indicated that all piles would have to be jetted and that bearing values greater than 40 tons per pile would be reached at a point elevation of about El. −30. The fill is a fine nonplastic sand weighing about 110 lb per cu ft. All of it passes a No. 8 screen; about 90% passes a 50-mesh screen; 20% passes a 100-mesh screen; and only 4% passes a 200-mesh screen. The outboard beach of the dike was on a slope of about 1 on 10 prior to the beginning of the work on the quay wall.

CONSTRUCTION PROCEDURE

The quay wall contractor first excavated a wide trench in the dike to about El. 1. He then jetted and drove the mat of wooden piles to a point elevation

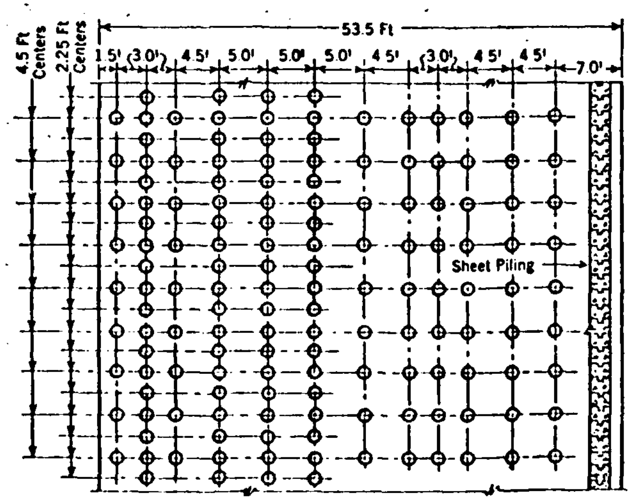


FIG. 54.—TYPICAL PILE PLAN

of about El. − 54. After this, he leveled off the top of the fill, placed a 4-in. layer of crushed stone, and cast all but the outer 9 ft of the concrete relieving platform. He then jetted and drove the concrete sheet piles to a point elevation of El. − 60 and anchored them back to the relieving platform. A general view of the construction is shown in Fig. 55.

At first it was planned to use steel sheet piles along the outer face of the wharf to retain the fill but, because of wartime conditions, they could not be obtained. Accordingly, concrete sheet piles, 21 in. deep and 19 in. wide, were

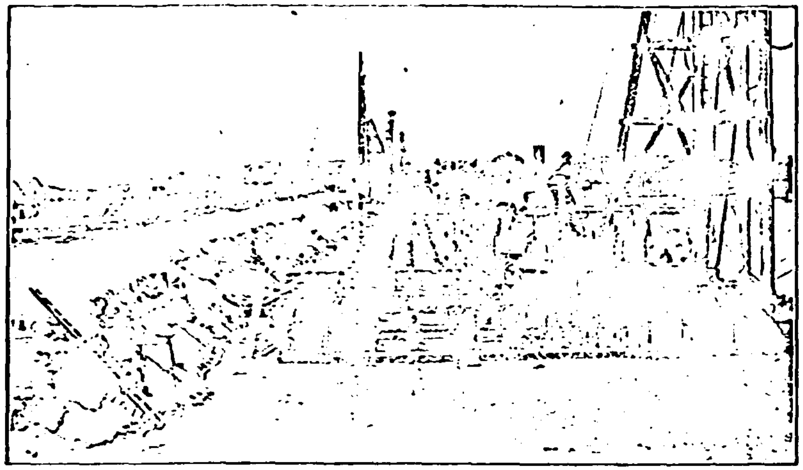


FIG. 55.—QUAY WALL UNDER CONSTRUCTION

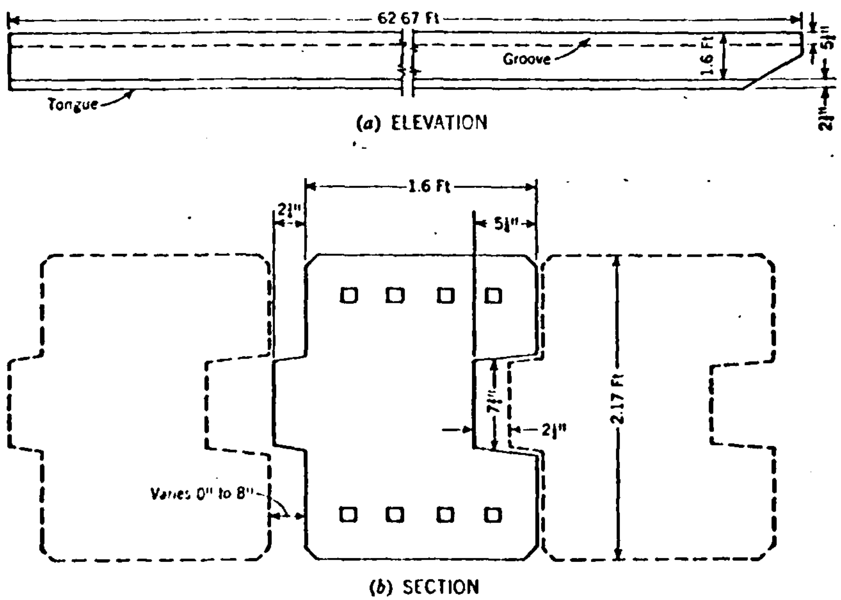
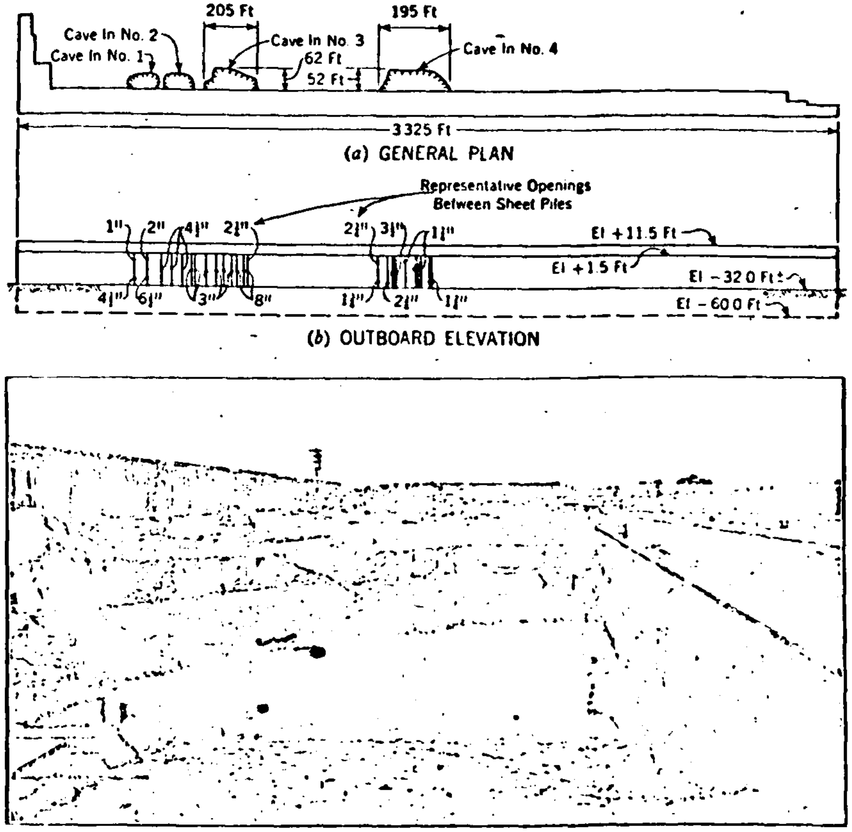


FIG. 56 DIMENSIONS OF CONCRETE SHEET PILE

used. The tongues projected 2¾ in. and the grooves were 5¼ in. deep. Thus, vertical holes were left between the piles, measuring about 2¾ in. by 7½ in. in plan as shown in Fig. 56. It was recognized that tight joints between the

sheet piles could not be expected in practice. The specifications, therefore, required that all joints be cleared of sand and silt, by a water jet, to a depth of 3 ft below the future dredge line—namely, to El. −43. The holes and spaces between the piles were then to be filled with cement grout pumped into place from El. −43 to the tops of the piles. To test the effectiveness of arrangement, two 30-ft sheet piles were driven so that their lower ends were about 10 in. apart with their tops in contact. Neat cement was forced into the groove



(c) VIEW OF CAVE-IN NO. 3

FIG. 57. LOCATION OF CAVE-INS BEHIND QUAY WALL

through a pipe whose lower end was raised as the grout rose around it. When the piles were pulled, it was found that the grout had effectively filled the gap between the piles. Therefore, it was decided to proceed with the use of neat cement grouting. Because the entire lengths of the sheet piles were in the ground, it was not possible to inspect the results of driving or grouting until after the material on the outside had been dredged.

To speed his work, the dredging contractor began by dredging to El. −35 in one cut right to the sheet piles, using a 30-in. suction dredge. He then intended to make a second cut to reach the required depth of 40 ft. As the dredging progressed, four large cave-ins occurred suddenly soon after the outside material was removed. The location of these failures, and a view of cave-in No. 3 are shown in Fig. 57. These cave-ins extended as much as 60 ft beyond the inner edge of the relieving platform, or more than 100 ft from the sheet piles. They varied in depth to 35 ft and in length to about 200 ft. The largest single cave in involved about 8,000 cu yd. It is impossible to determine the rate at which the material was lost since it is not known how long the loss may have been in progress under the relieving platform before the final cave-ins occurred. It appears however that as much as ±7,000 cu yd were lost in 24 hours, or an average of about 300 cu yd per hr. The peak rate is not known.

Inspections by divers showed that in about three hundred of the two thousand joints, the adjacent faces of the sheet piles did not meet properly by amounts varying from less than 1 in. to a maximum of about 8 in., measured at El. −30. These conditions were scattered along the entire length of the wall and there were numbers of openings extending through the wall where grout had failed to plug the joints effectively. In some cases, loose cylinders of grout the size of the grout pipe were found where the character of the bottom material had prevented proper intrusion of the grout.

The officer-in-charge was concerned about exposing the untreated wooden piles under the relieving platform to the action of marine borers and rot. Accordingly, he made a detailed exploration of the settlements that occurred under the entire length of the platform in locations other than the cave-ins and found that the fill had settled over wide areas from little or nothing to as much as 30 in. There was no evidence of any lateral translation of the relieving platform or of any part of the sheet piles. Mounds of sand found by divers outside the sheet piles had slopes as steep as 2 on 1. However, divers did not report seeing sand pouring through the openings. In one case, a hole to El. −44 was found outside the sheet piling in a region where the level of adjacent dredging was about 10 ft higher. It is not known whether this resulted from accidental overdredging or from a "blowout" under the ungrouted gaps between the piles below El. −43, or from both.

The dredging procedure in the second half of the job was then changed so that the first cut was made to El. −25, the second to El. −35, and the final cleanup cut to El. −40. Also, a small berm was left at the foot of the sheet piling to guard against overdredging, and the cutter head was kept at least 25 ft from the wall. The −25-ft level was used as a working platform from which divers grouted all suspicious looking joints. Wide joints that appeared dangerous were grouted practically to the tips of the piles before the dredging was extended to lower levels. With this revised procedure no further cave-ins occurred. The contractor finally pumped concrete sand under the slab so that at times the pavement back of the wall was heaved without, however, starting any further loss of material.

Discussion of Failure

It is not uncommon for a structurally adequate wall to fail to retain the backfill because of leaks through the sheet piling. Ordinarily, without the relieving platform, leaks can be detected early by the formation of small holes in the surface just behind the sheet piles. With the platform, there was no chance to see such holes if they formed. The cave-ins occurred suddenly and covered rather large areas. (In Fig. 53 the cave-in lines beneath the platform were assumed.)

The obvious answer, of course, is to make the sheet piles tight. Since this may not always be entirely practicable it is desirable to understand how material is lost through small openings, so that precautionary steps may be taken to provide appropriate safeguards. This appears to be especially important where sand dikes are relied on to reduce the lateral pressures of clay backfills, since a loss of dike material might then subject the bulkhead to dangerous lateral loads.

In this case a fine sand fill was placed hydraulically to its full height before any of the piles were driven. Large storehouses were built on spread footings on this type of fill with no noticeable settlement. Although the pile jetting operations probably loosened the sand, it is likely that the final driving of the last few feet of each pile, without jetting, recompacted it to some extent. No data are available to show what densities were actually obtained. Some of the sand that escaped was virgin bottom, not recent fill.

A number of explanations have been considered. A sudden jar caused by construction operations, or a sudden outward movement of the sheet piling, might cause a "quick" condition in the fine sand and make it behave temporarily as a heavy liquid mass. There was a suspicion that the cutterhead of the dredge struck the piling in one case. However, there is no suspicion of this in the other cases and no movements of the piles were observed. Therefore, it is not likely that the cutterhead of the dredge struck the piling since such an occurrence would not explain the fact that the loss of material continued for 24 hours. Another explanation considered was that, as sand slowly washed out between the sheet piles, cavities were formed behind the piles and these holes were maintained by the arching action of the sand. As the cavities enlarged and the loads become too great, the arches collapsed successively and built up pore pressures which liquefied the material and permitted it to flow out rapidly. The extent to which arched cavities could form would depend on the cohesion of the material in the backfill since ordinary cohesionless sand will not arch in this manner under water. This also is not a likely explanation even though such action may have occurred locally and contributed in a minor way to the loss of some material; nor is it likely that the phenomenon of internal erosion was involved—that is, the gradual formation of underground channels caused by percolating water escaping into a free opening through which material can be carried.

Where the grouting extended only a few feet below the dredged level, and the spaces between the piles were large enough, it is possible that the passive pressure on the outside was not great enough to resist the inside, active pressure.

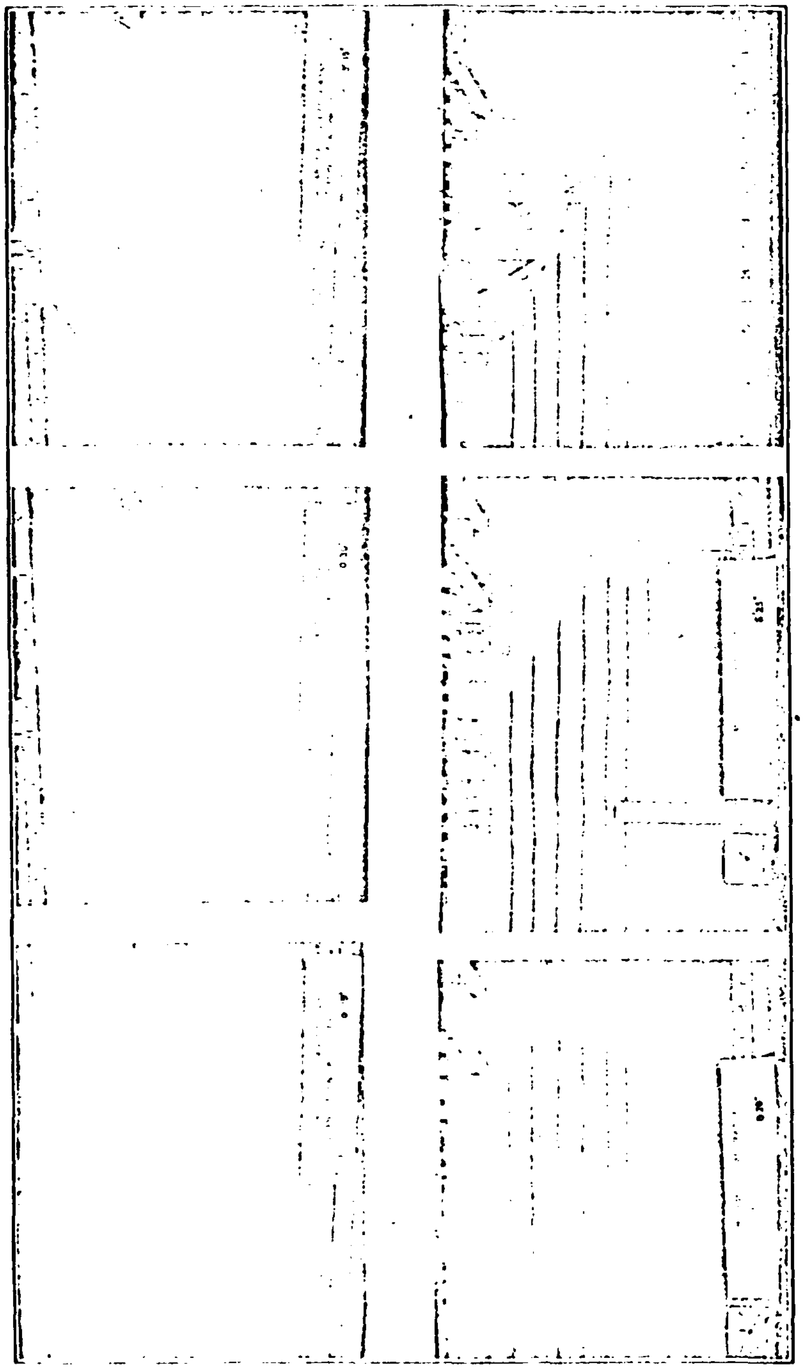


FIG. 58.—LABORATORY TESTS ON FINE DAYTONA BEACH SAND

This may have caused blowouts which then created quick conditions behind the bulkhead.

It is concluded, however, that most of the loss was started by the simple escape of cohesionless sand through openings between the piles above the dredged bottom. In an effort to visualize what might have happened, Professor Tschebotarioff ran a few laboratory demonstrations with a fine Daytona Beach sand. The tests were made in a glass sided tank 60 in. long by 24 in. high by 8 in. thick. A semicircular hole, 2 in. in diameter, was cut in the bulkhead against the glass side about 16 in. from the top.

In the first case the water was kept at the same level on both sides of the bulkhead and above the level of the sand. When the plug was removed, the cohesionless sand ran out of the hole by gravity. This quickly formed a vertical "pipe" of moving "quick" sand falling through water. The hole became enlarged within a few minutes. In the second case, the water was kept above the sand but was lowered on the outside. The action was the same but accelerated.

In the third case (see upper row, Fig. 58), water was kept slightly below the sand surface in back of the bulkhead and was kept somewhat lower on the outside. The action was similar to the other cases. In still another case (see lower row, Fig. 58), dowels were installed to simulate piles; again, the action was similar. The piles had little effect in retarding the rate of loss of the sand. It is possible that the reduction of the static pressure in the moving mass, as it acquires velocity, accelerates the loss of material from the banks, since the pressure in the banks would then be relatively greater. The escaping material was in a quick condition and the action would be expected to continue until the sand reached its natural angle of repose. In the prototype, it is entirely possible that chunks of material having some cohesion might have tumbled off the banks and caused a successive spread of localized quick conditions. Also, the wood piles could retard the loss of material having some cohesion and aid in building up higher and steeper banks than would otherwise be the case. This condition would tend to build up larger cavities and pressures and to produce sudden slides. The basic initial cause, however, appears to have been the simple flow of noncohesive material seeking its natural angle of repose under water.

The foregoing suggests that sandy materials used in backfills not only should have as great an angle of repose as practicable but also should be selected and placed so as to minimize the danger of loss from possible leaks in the bulkhead. The studies indicate the desirability of using densely graded coarse angular materials against the bulkhead to stop the loss of fines and to develop arching action across small openings. Of course, the thicker the sand dike behind the wall, the greater will be the factor of safety against damage resulting from unexpected leakage of the backfill.

SUMMARY

Although there is nothing particularly new in this case, it emphasizes certain points that is: (*a*) The importance of having tight walls sufficiently deep; (*b*) the need for guarding against conditions that can result in temporarily induced

pore pressures which, in turn, cause a loss of materials or excessive lateral pressures not contemplated in the design; (*c*) the fact that a dense mass of piles behind the bulkhead cannot be relied on to retard the loss of materials or to reduce lateral pressures especially if a quick condition is initiated; and (*d*) the material that is placed immediately in back of the bulkhead should be carefully selected and its placement controlled. Where a dike of selected material is relied on in the design to reduce the lateral pressures of other softer backfills, it is especially important to guard against its loss and to provide means for easy inspection and replacement of lost material. The Bureau of Yards and Docks now provides vertical 12-in. pipes between bents to facilitate inspection and replacing material under relieving platforms and pavements.

5.5 COST ESTIMATES - CARRIER QUAY WALL

ACTIVITY: CHESAPEAKE DIVISION WASHINGTON D.C.	SPECIFICATION NO.	A-E FIRM NAME: BLAYLOCK-WILLIS	SHEET 1 OF 3
PROJECT TITLE: REPAIRS TO CARRIER QUAYWALL NAVAL AIR STATION, NORTH ISLAND	ESTIMATED BY: STATUS:	CHECKED BY: Robert Whitelaw	DATE: September 24, 1984 J.O. NO.: 1644-06

DESCRIPTION	QUANTITY NUMBER	UNIT	MATERIAL COST UNIT COST	MATERIAL COST TOTAL	LABOR COST UNIT COST	LABOR COST TOTAL	ENGINEERING ESTIMATE UNIT COST	ENGINEERING ESTIMATE TOTAL
In the water costs are based on Contractor's bid unit utilizing following crew and equipment.								
2 Man Dive Crew in Water with Equipment								
Topside Tender and Standby Diver with Equipment Including Air Compressor								
TOTAL CREW RATE							$2,168/Day or $270/Hour	
METHOD 1								
Core 2" Diameter Holes at Pile Joint, 3/4 Hour Each	2,500	EA					203	507,500
Grout Wall Through Holes Material Plus Pump = 70/C.Y.	2,500	EA					70	175,000
Crew at 1 Hour / Hole	2,500	EA					88	220,000
Fill Leaking Joints	625	EA					123	76,900
Plug Holes Upon Completion	2,500	EA					50	125,000
Core 1 Row 4" Diameter Holes through Relieving Platform	129	EA					350	45,100
Grout from Surface Material Pump & Labor	750	CY					80	60,000
TOTAL METHOD 1								1,209,500

ACTIVITY: CHESAPEAKE DIVISION WASHINGTON D.C.	SPECIFICATION NO.	A-E FIRM NAME: BLAYLOCK-WILLIS	SHEET 2 OF 3
PROJECT TITLE: REPAIRS TO CARRIER QUAYWALL NAVAL AIR STATION, NORTH ISLAND	ESTIMATED BY: STATUS:	CHECKED BY: Robert Whitelaw	DATE: September 24, 1984 J.O. NO.: 1644-06

DESCRIPTION	QUANTITY NUMBER	QUANTITY UNIT	MATERIAL COST UNIT COST	MATERIAL COST TOTAL	LABOR COST UNIT COST	LABOR COST TOTAL	ENGINEERING ESTIMATE UNIT COST	ENGINEERING ESTIMATE TOTAL
METHOD 2								
From the surface, Contractor core through relieving platform two parallel rows of holes. Price includes tunnel ventilation, dewatering, clean up, alignment control and plugging after grouting.								
Core 2 Rows 4" Diameter Holes @ 25' O.C.	258	EA					350	90,300
Grout from Surface: Material, Pump and Labor	1,500	CY					80	120,000
Clean Pile Joints 1 Hour Each	600	EA					270	162,000
Sealing Material / Pile Joint at 9.60/Ft. x 28' = 269 / Joint	600	EA					269	161,400
Crew / Joint at 1 Hour Each	600	EA					270	162,000
TOTAL METHOD 2								695,700

ACTIVITY: CHESAPEAKE DIVISION WASHINGTON D.C.	SPECIFICATION NO.	A-E FIRM NAME: BLAYLOCK-WILLIS	SHEET 3 OF 3
PROJECT TITLE: REPAIRS TO CARRIER QUAYWALL NAVAL AIR STATION, NORTH ISLAND	ESTIMATED BY: STATUS:	CHECKED BY: Robert Whitelaw	DATE: September 24, 1984 J.O. NO.: 1644-06

DESCRIPTION	QUANTITY NUMBER	QUANTITY UNIT	MATERIAL COST UNIT COST	MATERIAL COST TOTAL	LABOR COST UNIT COST	LABOR COST TOTAL	ENGINEERING ESTIMATE UNIT COST	ENGINEERING ESTIMATE TOTAL
METHOD 3								
From the surface Contractor core through relieving platform as with Method 2.								
Core 2 Rows 4" Diameter Holes @ 25' O.C.	258	EA					350	90,000
Grout From the Surface: Material, Pump and Labor	1,500	CY					80	120,000
Demolition and Removal of Existing Buttresses on Face Quaywall	752	CY					100	75,000
PZ 27 Sheet Piles: Move In and Out		LS						10,000
Buy and Drive 50 Ft. Lengths	4,218,700	LB					.86	3,628,000
Pile Cap Connection to Face of Wall, Sandblast, Dowel, Epoxy, Reinforce	2,900	CY					272	789,000
Cementitious Fill	11,460	CY					61	699,000
TOTAL METHOD 3								5,411,000

5.6 TABLES OF STRUCTURAL ASSESSMENT

TABLE 5.1

RECORD OF STRUCTURAL ASSESSMENT

OLD AIR STATION BULKHEAD

DESCRIPTION OF QUAYWALL CONDITION					
QUAYWALL		TOP	MIDDLE	BOTTOM	STRUCTURAL COMMENTS
Station	Type				
0+70	Concrete	-------	-------	Soft	Gunite along wall above wale
1+46	Concrete	-------	-------	Soft	Gunite along wall above wale
3+46	Concrete	Soft	-------	Soft	Gunite along wall above wale
5+46	Concrete	Very Soft	-------	Very Soft	Heavy leak @ Station 3+96. Broken tie back @ Station 4+86 Gunite along wall above wale
7+46	Concrete	Soft	-------	Soft	Three heavy leaks Gunite along wall above wale
9+46	Concrete	Soft	-------	Soft	Some heavy leaks Heavy leak with some subsidence Gunite along wall above wale
11+46	Concrete	-------	-------	Soft	Subsidence behind wall @ Station 11+46 to 11+96. Rubble along wall @ Sta 11+96. Gunite along wall above wale.
13+46	Concrete	-------	-------	-------	Gravity wall from Station 12+33 to 12+93. Gunite along wall above wale
15+46	Concrete	-------	-------	-------	Gunite along wall above wale
17+46	Concrete	-------	-------	Very Soft	Tie-back wall stops @ Station 16+68
19+46	Concrete	-------	-------	Very Soft	Corner @ Station 18+61 is cracked & patched but still open.
21+46	Concrete	-------	-------	Soft	Vertical cracks @ Stations 20+92 & 21+42. The cracks are apprx. Centered between construction joints.
23+46	Concrete	-------	-------	Soft	Construction joint key cracked @ Station 22+06
25+46	Concrete	-------	-------	Soft	

TABLE 5.1

RECORD OF STRUCTURAL ASSESSMENT

OLD AIR STATION BULKHEAD

DESCRIPTION OF QUAYWALL CONDITION					
QUAYWALL		TOP	MIDDLE	BOTTOM	STRUCTURAL COMMENTS
Station	Type				
27+46	Concrete	--------	--------	Soft	
29+46	Concrete	--------	--------	Soft	
31+46	Concrete	--------	--------	Soft	
33+46	Concrete	--------	--------	Soft	
35+46	Concrete	--------	--------	Soft	
37+46	Concrete	--------	--------	Very Soft	
39+46	Concrete	--------	--------	Soft	
41+46	Concrete	--------	--------	Soft	
43+46	Concrete	-------	-------	-------	Asphalt rubble and rip-rap along face of wall
45+46	Concrete	-------	-------	-------	Rubble along face of wall
45+80	Concrete	--------	--------	Soft	

TABLE 5.2

RECORD OF STRUCTURAL ASSESSMENT

CARRIER QUAY WALL

DESCRIPTION OF QUAYWALL CONDITION					
QUAYWALL		TOP	MIDDLE	BOTTOM	STRUCTURAL COMMENTS
Station	Type				
0+50	Concrete	-------	-------	Firm	Slight leakage
2+00	Concrete	Soft	Firm	Soft	Slight leakage
4+00	Concrete	Soft	Soft	Firm	Slight to moderate leakage
6+00	Concrete	Soft	Soft	Firm	Moderate leakage with occasional heavy leakage
8+00	Concrete	Soft	Soft	Firm	Slight leakage heavy leak @ Station 6+70
10+00	Concrete	Soft	Soft	Soft	Slight to moderate leakage
12+00	Concrete	Soft	Firm	Firm	Slight leakage
14+00	Concrete	Soft	Soft	Soft	Slight to moderate leakage heavy leak @ Station 14+00
16+00	Concrete	Soft	Firm	Soft	Slight leakage with occasional moderate leakage, two heavy leaks
18+00	Concrete	Soft	Firm	Firm	Slight leakage with occasional moderate leakage, heavy leak @ Station 17+00
20+00	Concrete	Soft	Firm	Firm	Very slight leakage
22+00	Concrete	Firm	Soft	Firm	Very slight leakage
24+00	Concrete	Soft	Firm	Firm	Slight to moderate leakage
26+00	Concrete	Firm	Firm	Firm	Slight leakage

TABLE 5.2

RECORD OF STRUCTURAL ASSESSMENT

CARRIER QUAY WALL

DESCRIPTION OF QUAYWALL CONDITION					
QUAYWALL		TOP	MIDDLE	BOTTOM	STRUCTURAL COMMENTS
Station	Type				
28+00	Concrete	Soft	Soft	Firm	Very slight leakage
30+00	Concrete	Firm	Firm	Firm	Slight leakage
32+00	Concrete	Firm	Firm	Firm	Slight to moderate leakage
33+05	Concrete	Soft	Firm	Firm	Slight leakage
33+94	Concrete	-------	-------	Firm	No leakage

TABLE 5.3

RECORD OF STRUCTURAL ASSESSMENT

CARRIER QUAY WALL

DESCRIPTION OF QUAYWALL CONDITION - RELIEVING PLATFORM VOIDS		
QUAYWALL STATIONS	VOIDS * (inches)	STRUCTURAL COMMENTS
0+30		Manhole
0+74		Standpipe - clogged
0+98		Standpipe - cap frozen
1+22		Standpipe - clogged
1+49		Standpipe - cap frozen
1+75		Standpipe - clogged
1+92		Manhole
1+99	7	Standpipe
2+37		Standpipe - clogged
2+75		Standpipe - wooden plug
2+99		Standpipe - wooden plug
3+15		Access hole
3+24		Standpipe - wooden plug
3+49		Standpipe - clogged, cap missing

*Measured from the bottom of the relieving platform through standpipes located in utility tunnel.

TABLE 5.3

RECORD OF STRUCTURAL ASSESSMENT

CARRIER QUAY WALL

DESCRIPTION OF QUAYWALL CONDITION - RELIEVING PLATFORM VOIDS		
QUAYWALL STATIONS	VOIDS (inches)	STRUCTURAL COMMENTS
3+67		Manhole
3+73		Standpipe - clogged
3+98		Standpipe - clogged
4+25		Standpipe - wooden plug
4+49		Standpipe - clogged
4+74		Standpipe - clogged
5+06		Standpipe - clogged
5+25		Standpipe - clogged
5+49		Standpipe - clogged
5+75		Standpipe - clogged
6+00		Standpipe - clogged
6+06		Access Hole
6+24		Standpipe - clogged
6+50	4	Standpipe

TABLE 5.3

RECORD OF STRUCTURAL ASSESSMENT

CARRIER QUAY WALL

DESCRIPTION OF QUAYWALL CONDITION - RELIEVING PLATFORM VOIDS		
QUAYWALL STATIONS	VOIDS (inches)	STRUCTURAL COMMENTS
6+67		Manhole
6+75		Standpipe - broken off and plugged
6+82		Access hole
7+01		Standpipe - clogged
7+24		Standpipe - clogged
7+48		Standpipe - clogged
7+72		Standpipe - clogged
7+96		Standpipe - clogged
8+19		Standpipe - wooden plug
8+43		Standpipe - bent with wooden plug
8+77		Standpipe - clogged
9+02		Standpipe - clogged
9+27		Standpipe - clogged
9+51		Standpipe - clogged

TABLE 5.3

RECORD OF STRUCTURAL ASSESSMENT

CARRIER QUAY WALL

DESCRIPTION OF QUAYWALL CONDITION - RELIEVING PLATFORM VOIDS		
QUAYWALL STATIONS	VOIDS (inches)	STRUCTURAL COMMENTS
9+67		Manhole
9+74		Standpipe - clogged
10+00		Standpipe - wooden plug
10+26		Standpipe - clogged
10+51		Standpipe - clogged
10+75		Standpipe - clogged
11+01		Standpipe - clogged
11+18		Manhole
11+27		Standpipe - clogged
11+49		Standpipe - clogged
11+76		Standpipe - clogged
11+80		Access hole
12+01		Standpipe - clogged
12+26		Standpipe - clogged

TABLE 5.3

RECORD OF STRUCTURAL ASSESSMENT

CARRIER QUAY WALL

DESCRIPTION OF QUAYWALL CONDITION - RELIEVING PLATFORM VOIDS		
QUAYWALL STATIONS	VOIDS (inches)	STRUCTURAL COMMENTS
12+50		Standpipe - clogged
12+68		Manhole
12+75		Standpipe - clogged
13+01		Standpipe - clogged
13+06		Access hole
13+27		Standpipe - clogged
13+51		Standpipe - clogged
13+76		Standpipe - clogged
14+01		Standpipe - clogged
14+26		Standpipe - clogged
14+49		Standpipe - clogged
14+76		Standpipe - clogged
15+01		Standpipe - clogged
15+05		Access hole

TABLE 5.3

RECORD OF STRUCTURAL ASSESSMENT

CARRIER QUAY WALL

DESCRIPTION OF QUAYWALL CONDITION - RELIEVING PLATFORM VOIDS		
QUAYWALL STATIONS	VOIDS (inches)	STRUCTURAL COMMENTS
15+27		Standpipe - clogged
15+52		Standpipe - clogged
15+68		Manhole
15+78		Standpipe - missing
16+26		Standpipe - wooden plug
16+51		Standpipe - clogged
16+76		Standpipe - clogged
16+89		Access hole
17+01	11	Standpipe
17+27	1½	Standpipe
17+51		Standpipe - clogged
17+76		Standpipe - clogged, cap missing
18+00		Standpipe - wooden plug
18+26		Standpipe - wooden plug

TABLE 5.3

RECORD OF STRUCTURAL ASSESSMENT

CARRIER QUAY WALL

DESCRIPTION OF QUAYWALL CONDITION - RELIEVING PLATFORM VOIDS		
QUAYWALL STATIONS	VOIDS (inches)	STRUCTURAL COMMENTS
18+51		Standpipe - wooden plug
18+58		Access hole
18+68		Manhole
18+74		Standpipe - missing
19+00		Standpipe - clogged
19+26		Standpipe - cap frozen
19+51		Standpipe - clogged
19+76		Standpipe - clogged
20+01		Standpipe - clogged
20+26		Standpipe - clogged
20+49		Standpipe - clogged
20+76		Standpipe - clogged
20+83		Access hole
21+00		Standpipe - clogged

TABLE 5.3

RECORD OF STRUCTURAL ASSESSMENT

CARRIER QUAY WALL

DESCRIPTION OF QUAYWALL CONDITION - RELIEVING PLATFORM VOIDS		
QUAYWALL STATIONS	VOIDS (*inches*)	STRUCTURAL COMMENTS
21+26		Standpipe - clogged
21+50		Standpipe - clogged
21+68		Manhole - spalling at ceiling
21+76		Standpipe - bent
21+99		Standpipe - clogged
22+25		Standpipe - clogged
22+51		Standpipe - clogged
22+76		Standpipe - clogged
22+99	7	Standpipe
23+25		Standpipe - clogged
23+45		Standpipe - missing
23+74		Standpipe - clogged
23+98		Standpipe - clogged Spalling with exposed reinforcing at ceiling
24+21		Standpipe - clogged

TABLE 5.3

RECORD OF STRUCTURAL ASSESSMENT

CARRIER QUAY WALL

DESCRIPTION OF QUAYWALL CONDITION - RELIEVING PLATFORM VOIDS		
QUAYWALL STATIONS	VOIDS (inches)	STRUCTURAL COMMENTS
24+45		Standpipe - clogged
24+68		Manhole - spalling at ceiling
24+69		Standpipe - clogged
25+25		Standpipe - clogged
25+30		Access hole
25+72		Standpipe - clogged
26+00		Standpipe - cap frozen
26+26		Standpipe - clogged
26+75	7	Standpipe
26+98		Standpipe - clogged
27+03		Access hole
27+30		Standpipe - no void, very loose sand
27+44		Sandpipe - clogged
27+68		Manhole

TABLE 5.3

RECORD OF STRUCTURAL ASSESSMENT

CARRIER QUAY WALL

DESCRIPTION OF QUAYWALL CONDITION - RELIEVING PLATFORM VOIDS		
QUAYWALL STATIONS	VOIDS (*inches*)	STRUCTURAL COMMENTS
28+29	*3*	*Standpipe*
28+54	*2*	*Standpipe*
28+76		*Standpipe - clogged*
29+01		*Standpipe - clogged*
29+26		*Standpipe - clogged*
29+51		*Standpipe - clogged*
29+73		*Access hole*
29+76		*Standpipe - clogged*
30+02	*1*	*Standpipe*
30+26		*Standpipe - clogged*
30+51		*Standpipe - clogged*
30+68		*Manhole*
30+76		*Standpipe - clogged*
31+00		*Standpipe - clogged*

TABLE 5.3

RECORD OF STRUCTURAL ASSESSMENT

CARRIER QUAY WALL

DESCRIPTION OF QUAYWALL CONDITION - RELIEVING PLATFORM VOIDS		
QUAYWALL STATIONS	VOIDS (inches)	STRUCTURAL COMMENTS
31+26		Standpipe - clogged
31+52		Standpipe - clogged
31+76		Standpipe - clogged
31+77		Access hole
31+93		Manhole
32+01		Standpipe - clogged
32+25	13	Standpipe - soft sand
32+50		Standpipe - clogged
32+75		Standpipe - clogged
32+88		Manhole

TABLE 5.4

RECORD OF STRUCTURAL ASSESSMENT

PIER BRAVO

DESCRIPTION OF PILE CONDITION				
PILE	TOP	MIDDLE	BOTTOM	STRUCTURAL COMMENTS
1J	Firm	Soft	Firm	
2N	Soft	Soft	Soft	
3M	Firm	Firm	Soft	
4K	Firm	Firm	Firm	
4R	Soft	Firm	Firm	
5J	Firm	Firm	Firm	
6M	Firm	Firm	Firm	
6P	Firm	Firm	Firm	
7R	Firm	Firm	Soft	
8K	Soft	Soft	Firm	
9R	Firm	Firm	Firm	
10E	Soft	Firm	Firm	
11J	Firm	Firm	Firm	
12B	-------	-------	Firm	
14D	Firm	Firm	Firm	

TABLE 5.4

RECORD OF STRUCTURAL ASSESSMENT

PIER BRAVO

DESCRIPTION OF PILE CONDITION				
PILE	TOP	MIDDLE	BOTTOM	STRUCTURAL COMMENTS
14F	Soft	Firm	Firm	
15M	Firm	Soft	Firm	
17C	Firm	-------	Firm	
17E	Firm	Firm	Firm	
18G	Firm	Firm	Firm	
19H	Soft	Firm	Firm	
19R	Firm	Firm	Firm	
20B	--------	-------	Firm	
21N	Firm	Firm	Firm	
22H	Soft	Firm	Firm	
22M	Soft	Soft	Firm	
22R	Firm	Firm	Firm	
23H	Firm	Firm	Firm	
24M	Soft	Soft	Firm	
25H	Firm	Firm	Firm	

TABLE 5.4

RECORD OF STRUCTURAL ASSESSMENT

PIER BRAVO

DESCRIPTION OF PILE CONDITION				
PILE	TOP	MIDDLE	BOTTOM	STRUCTURAL COMMENTS
25P	Firm	Firm	Firm	
26N	Firm	Firm	Firm	
27H	Firm	Firm	Firm	
27L	Firm	Firm	Firm	
28H	Firm	Firm	Firm	
29K	Soft	Firm	Firm	
29P	Soft	Firm	Firm	
31D	Firm	Firm	Firm	
32B	Firm	Firm	Firm	
33F	Firm	Firm	Firm	
35F	Soft	Soft	Soft	
36A	Firm	Firm	Firm	
36H	Firm	Firm	Firm	
37B	Soft	Soft	Soft	
38D	Firm	Firm	Firm	

TABLE 5.4

RECORD OF STRUCTURAL ASSESSMENT

PIER BRAVO

DESCRIPTION OF PILE CONDITION				
PILE	TOP	MIDDLE	BOTTOM	STRUCTURAL COMMENTS
39C	Firm	Firm	Firm	
39G	Firm	Firm	Firm	
42C	Firm	Soft	Soft	
42J	Firm	Firm	Firm	
131D	Soft	Soft	Soft	
132B	Soft	Soft	Soft	
133G	Firm	Soft	Soft	
136F	Soft	Soft	Soft	
136J	Firm	Soft	Soft	
137B	Soft	Firm	Firm	
138D	Soft	Soft	Firm	
139E	Firm	Soft	Firm	
139H	Firm	Firm	Firm	
142A	Soft	Firm	Firm	
142J	Soft	Soft	Soft	

TABLE 5.5

RECORD OF STRUCTURAL ASSESSMENT

PIER JK

DESCRIPTION OF PILE CONDITION - LEVEL I ONLY				
PILE	TOP	MIDDLE	BOTTOM	STRUCTURAL COMMENTS
9-D				Very soft
9-E				Very soft
9-F				Very soft
9-H				Very soft - terrible shape
12-B				Very soft - bad shape
13-B				Very soft - bad shape
13-H				Very soft
18-K				Very soft Soft to corner reinforcing
18-L				Very soft Soft to corner reinforcing
18D-N				Very soft Soft to corner reinforcing
18D-P				Very soft Soft to corner reinforcing
18D-R, T V,X,Z,BB				Soft
19-B,C,D E,F,G				Firm with occasional soft
21D-K,L,N P,R,T,V,X Z,BB				Soft
B-1 to 13				Soft

TABLE 5.5

RECORD OF STRUCTURAL ASSESSMENT

PIER JK

DESCRIPTION OF PILE CONDITION - *LEVEL I ONLY*				
PILE	TOP	MIDDLE	BOTTOM	STRUCTURAL COMMENTS
E-1 to 13				*Soft to very soft*
H-1 to 13				*Soft to very soft*

5.7 STRUCTURAL CALCULATIONS

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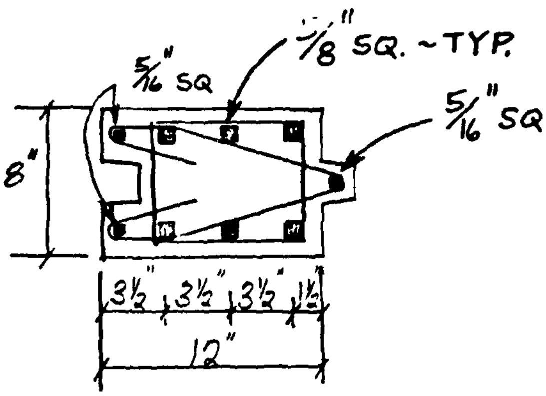


$A_s = 3(0.31) = 0.93\ in^2$

$F_y = 40$ ksi $\quad f_s = 20000$ psi

$f'_c = 2000$ psi $\quad f_c = 900$ psi

$n = 11 \quad k = 0.331 \quad j = 0.889$

$K_b = 132$

$b = 12 - 2 = 10$ IN.

$d = 8 - 1\frac{1}{2} = 6.5$ IN.

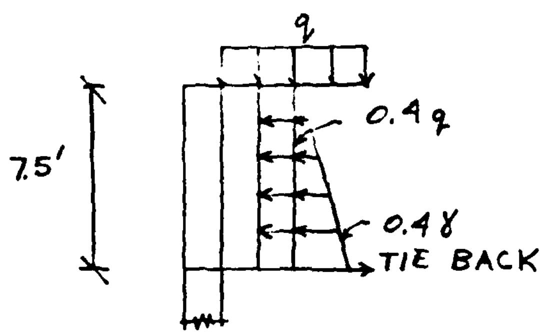
$\rho = \frac{A_s}{bd} = \frac{0.93}{10(6.5)} = 0.014$

$F = \frac{bd^2}{12000} = 0.035 \quad K = 163$

MAXIMUM MOMENT

$M = FK = 163(0.035) = 5.71$ ft-k $= 5710$ ft-lb

MAXIMUM SURCHARGE



q = SURCHARGE

$\gamma = 120$ PCF

$$M = \left[0.4q\,(7.5)^2(\tfrac{1}{2}) + 0.4(120)(7.5)^3(\tfrac{1}{2})(\tfrac{1}{3})\right](1)$$

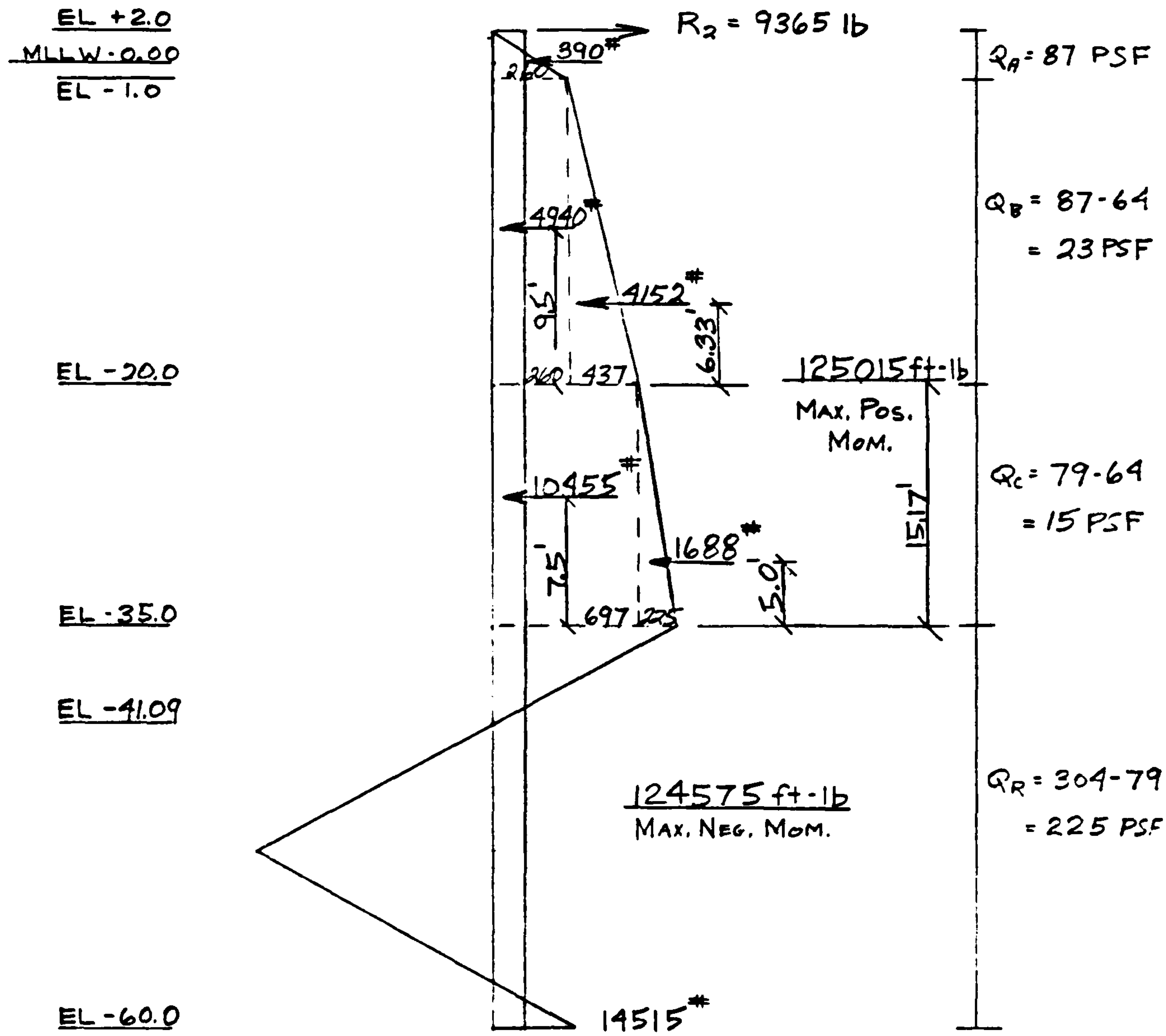
$$= 11.25\,q + 3375$$

$M_{max} = 5710$ ft-lb

so $q_{max} = \frac{5710 - 3375}{11.25} = 208$ PSF

MAXIMUM SURCHARGE - 208 PSF

LOADING DIAGRAM

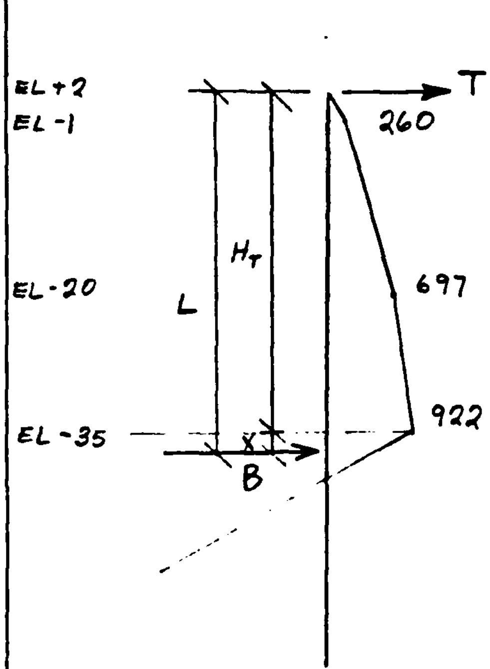


REFERENCE:

ELEVENTH NAVAL DISTRICT, SAN DIEGO, CALIFORNIA
NAVAL AIR STATION
QUAY WALL AT SPANISH BIGHT
SHEET 2 OF 14, SPECIFICATION No 13977
P.W. DRAWING No. NA11/N15-3(9)
DATED: JUNE 6, 1944

DESIGN FORCES

EQUIVALENT BEAM METHOD



$\phi = 36^\circ$

so $x = 0.03 H_T = 0.03(37) = 1.11'$

$L = 37 + 1.11 = 38.1'$

$$W = \frac{260(3)}{2} + \frac{(260+697)(19)}{2} + \frac{(697+922)(15)}{2} + \frac{(922+672)(1.1)}{2} = 22500 \text{ lb}$$

$$M = \frac{WL}{8} = \frac{22500(38.1)}{8} = 107160 \text{ ft-lb}$$

$$\Sigma M_T = 390(2) + 4940(9.5) + 4152(15.67) + 10455(29.5) + 1688(32) + 739(37.6) + 138(37.4) - 38.1B = 0$$

$\underline{B = 13337 \text{ lb}}$

$\underline{T} = 22500 - 13337 = \underline{9162 \text{ lb}}$

CALC. LOCATION OF M_{MAX}

$9162 - 390 - 260x - 23x^2(1/2) = 0$

$x = 18.54' \Rightarrow EL -19.54'$

$M_{MAX} = 9162(21.54) - 390(19.5) - 4820(9.27) - 3953(6.18)$

$\underline{+M_{MAX} = 120634 \text{ ft-lb}}$

$t = x + 1.2\sqrt{6B/(K_p - K_a)\gamma'}$

$t = 1.1 + 1.2\sqrt{6(13337)/3.69(60)}$

$t = 23.93'$

NOTE: REACTIONS AND MOMENTS ARE WITHIN 4 PERCENT OF THE VALUES SHOWN ON THE LOADING DIAGRAM.

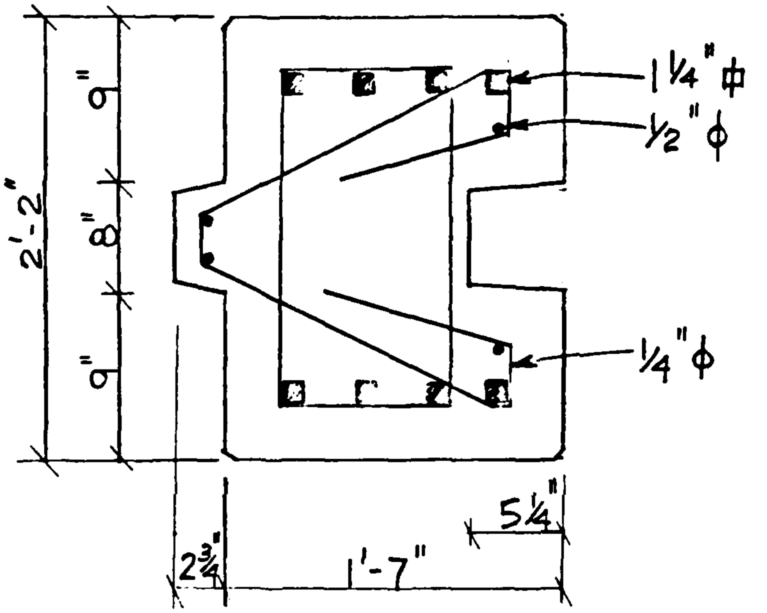
MIN REQ'D PILE LENGTH

$L_{TOT} = 37 + 23.93 = 60.9'$

$\underline{60.9' < 62' \quad OK}$

BENDING STRESSES

ASSUME: $f'_c = 4000$ psi $F_y = 40$ ksi



$A_c = 474 \text{ in}^2$

$b = \frac{474}{26} = 18.2''$

$A_s = 2(0.20) + 4(1.25)^2$
$= 6.65 \text{ in}^2$

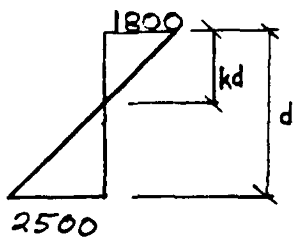
$A_s d = 6.25(22.375) + 0.20(19.25) + 0.2(14.75)$
$= 146.64 \text{ in}^3$

$d = \frac{146.64}{6.65} = 22.05''$

$M = 125015$ ft-lb $= 1500180$ in-lb/ft

$M = \frac{20}{12}(1500180) = 2500300$ in-lb

kd BALANCED



$f_c = 1800$ psi $E_c = 3.60 \times 10^6$ psi
$f_y = 20000$ psi $E_s = 29 \times 10^6$ psi
$n = 8.0$

$\frac{kd}{d} = \frac{1800}{1800 + 2500} \Rightarrow kd = 0.419\,d$

@ $d = 22.05''$ $kd = 9.23''$

DATE 10-1-84

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SHEET NO. 44

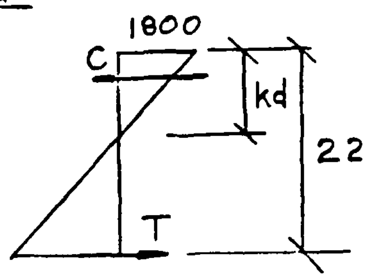
ENGINEERS TES

JOB NO.

SUBJECT CARRIER QUAYWALL

BENDING STRESSES (CONT)

kd



$M = 2500300$ IN-LB

$C = 1800(kd)(18.2)(1/2) = 16380\,kd$

$M = C(22.05 - 1/3\,kd)$

$2500300 = 16380\,kd\,(22.05 - 1/3\,kd)$

$kd = 7.86''$, $jd = 19.43''$

STRESS

SINCE $kd < kd_{BAL}$

$f_s = 20000$ psi

$$f_c = \frac{20000(7.86)}{8(22.05-7.86)} = 1385 \text{ psi}$$

$f_s = 20000$ psi
$f_c = 1385$ psi

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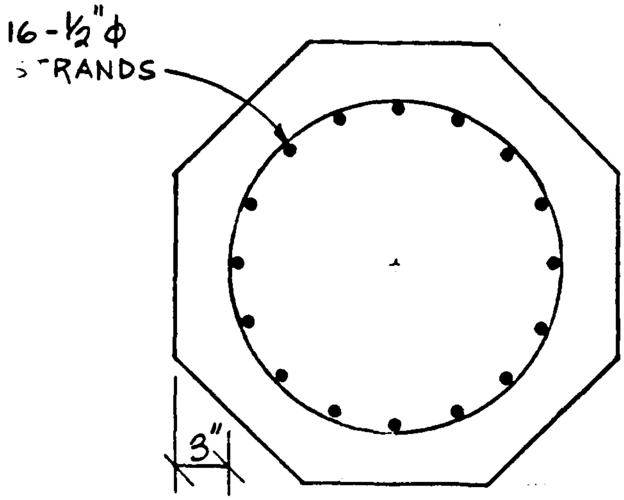
PILE CAPACITIES

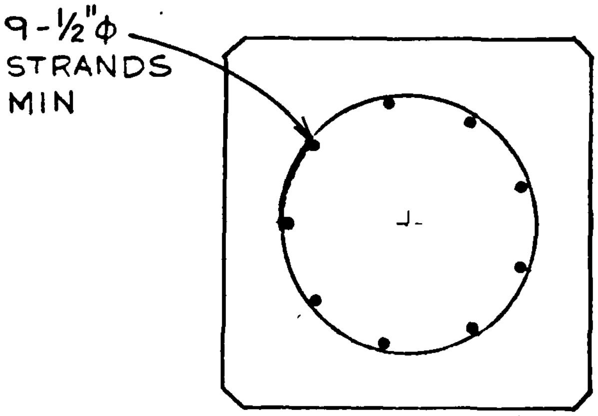
24" OCTAGONAL



LIVE LOAD - 600 PSF

LOAD CAPACITY - 276 TONS

20" SQUARE



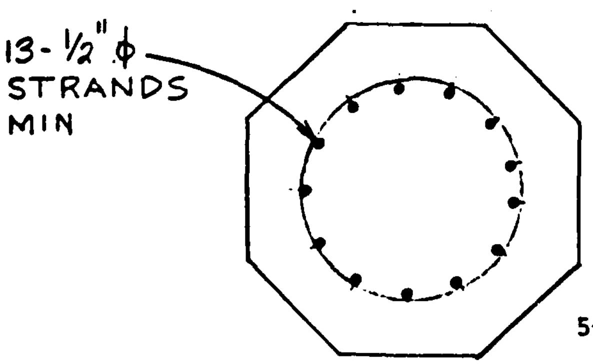
LIVE LOAD - 150 PSF

f_{pc} = 800 PSI

f_{pu} = 270 KSI (ASSUMED)

f'_c = 6000 PSI

18" OCTAGONAL



LIVE LOAD - 150 PSF

f_{pc} = 800 psi

f_{pu} = 270 KSI (ASSUMED)

f'_c = 6000 PSI

DATE 10-1-84

Blaylock-Willis and Associates.
Engineers
1909 McKee St., San Diego, CA 92110-1998

SHEET NO. 2/5

ENGINEERS TES

JOB NO.

SUBJECT PIER BRAVO

LOADS

24" OCTAGONAL

	PSF
10" CONC. TOPPING	125
T BEAM $\left(\frac{1(3)}{6}+\frac{5.5}{12}\right)150$	144
PILE CAP $\left(\frac{2(5)(1)}{31}\right)150$	48
	317
LIVE LOAD	600
TOTAL LOAD	917

TRIB AREA = 31(15) = 465 FT^2

P_{TOT} = 917(465) + 495(86) = 469000 lb

= 469 KIPS = 234 TONS

20" SQUARE & 18" OCTAGONAL

	WALKWAY PSF	MOORING PLATFORM PSF
CONC TOPPING $\frac{4(150)}{12}=50$	50	50
T BEAM $\frac{8.1(115)}{12(3)}+\frac{2(115)}{12}=45$	45	—
CONC SLAB $\frac{8(115)}{12}=77$	—	77
PILE CAP $\frac{3(3)(150)}{6(45)}=5$; $\frac{3(3)(150)}{15}=90$	5	90
	100	217
LIVE LOAD	150	150
	250	367
TRIB. AREA	270 FT^2	248 FT^2
P_A	67.5 k	91 k
P_{PILE} · 76.5(280) = 21.4 k 86.5(417) = 36 k	21.4 k	36 k
P_{TOT}	89 k	127 k

P_{TOT} = 89 KIPS = 45 TONS

P_{TOT} = 127 KIPS = 64 TONS

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1909 McKee St., San Diego, CA 92110-1998

DATE 10-1-84
ENGINEERS TES
SUBJECT PIER BRAVO
SHEET NO. 3/5
JOB NO.

PILE CAPACITIES

24" OCTAGONAL

$f_{pu} = 270$ ksi
$f_{pc} = 700$ PSI (ASSUMED)
$f'_c = 5000$ PSI (ASSUMED)

$A_s = 16(0.153)$
$A = 477$ IN^2
$I = 18180$ IN^4
$S = 1515$ IN^3
$r = 6.17$ IN

$$R = 1.32 - \frac{0.006\, l_u}{r} \qquad l_u = 0.50(69) = 34.5'$$

$$R = 0.92$$

$$F_a = [0.33 f'_c - 0.27 f_{pc}]R$$
$$= [0.33(5000) - 0.27(700)]\, 0.92 = 1344 \text{ PSI}$$

$$F_B = [0.40 f'_c - f_{pc}]R$$
$$= [0.40(5000) - 700]\, 0.92 = 1196 \text{ PSI}$$

$$f_a = P/A \qquad f_b = M/S = Pe/S$$

$$\frac{f_a}{F_a} + \frac{f_b}{F_B} \le 1.0 \quad \text{(CONSERVATIVE)}$$

$$\frac{P}{477(1344)} + \frac{P(0.05)(24)}{1515(1196)} = 1.0$$

$P_{max} = 450$ KIPS $= 225$ TONS

225 TONS $\not\geq$ 234 TONS 1.04 % CLOSE ENOUGH

DATE 10-1-84

ENGINEERS TES

SUBJECT PIER BRAVO

Blaylock-Willis and Associates.
Engineers
1909 McKee St., San Diego, CA 92110-1998

SHEET NO. 4/5

JOB NO.

PILE CAPACITIES

20" SQUARE

$f_{pu} = 270$ KSI $\quad A_s = 9(0.153) = 1.38\ in^2$

$f_{pc} = 800$ PSI $\quad E_s = 28 \times 10^6$ PSI

$f'_c = 6000$ PSI $\quad A = 400\ in^2$

$I = 13333\ IN^4$

$R = 1.23 - \dfrac{0.008\,l_u}{r}$ $\quad l_u = 0.75(69) = 52'$ $\quad r = 5.77\ in$

(FIXED - PINNED)

$R = 0.36$

$F_a = [0.33 f'_c - 0.27 f_{pc}] R$

$= [0.33(6000) - 0.27(800)] 0.36 = 635$ psi

$F_b = [0.40 f'_c - f_{pc}] R$

$= [0.40(6000) - 800] 0.36 = 576$ psi

$f_a = P/A$, $f_b = M/S = Pec/I$

$\dfrac{f_a}{F_a} + \dfrac{f_b}{F_b} \le 1.0$ (CONSERVATIVE)

$\dfrac{P}{400(635)} + \dfrac{P(0.05)(20)(10)}{13333(576)} = 1.0$

$P_{MAX} = 191$ KIPS $= 95$ TONS

95 TONS > 64 TONS OK

PILE CAPACITIES

18" OCTAGONAL

$f_{pu} = 270$ KSI

$f_{pc} = 800$ PSI

$f'_c = 6000$ PSI

$A_s = 13(0.153) = 1.99$ IN^2

$A = 324$ IN^2

$I = 8748$ IN^4

$S = 972$ IN^3

$r = 5.20$ IN

$$R = 1.23 - \frac{0.008\,\ell_u}{r}$$

$\ell_u = 0.75(69) = 52'$ (FIXED - PINNED)

$R = 0.27$

$$F_a = [0.33 f'_c - 0.27 f_{pc}]R$$

$$= [0.33(6000) - 0.27(800)]\,0.27 = 476 \text{ PSI}$$

$$F_b = [0.40 f'_c - f_{pc}]\,R$$

$$= [0.40(6000) - 800]\,0.27 = 432 \text{ PSI}$$

$f_a = P/A \qquad f_b = M/S = Pe/S$

$$\frac{f_a}{F_A} + \frac{f_b}{F_b} \leq 1.0 \quad \text{(CONSERVATIVE)}$$

$$\frac{P}{324(476)} + \frac{P(0.05)(18)}{972(432)} = 1.0$$

$P_{MAX} = 116$ KIPS $= 58$ TONS

58 TONS > 45 TONS OK

5.8 BIBLIOGRAPHY

BIBLIOGRAPHY

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